DETERMINING THE FLOOD EFFECTS OF UNDOCUMENTED STOPBANKS WITHIN THE WAIMEA FLOODPLAIN

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ABSTRACT

Across the globe, floods are the most damaging natural hazard both in terms of the number of people affected and the economic loss. This is also the case within New Zealand due to many towns and cities being located on floodplains that are at risk of inundation (Rouse, 2012). Flood levees ("stopbanks" in New Zealand) aim to prevent inundation within these communities. The maintenance of the stopbanks is generally the responsibility of the local and regional councils. However, along with council-maintained stopbanks, numerous privately-owned or unconsented stopbank structures exist.

The Tasman District Council recognises this and acknowledges that several privately-owned or undocumented stopbanks exist within their jurisdiction. These undocumented stopbanks are not part of the Council's assets, meaning there is limited documentation of their physical properties, such dimensions, age, purpose and flood design capacity. As a result of this, the undocumented stopbanks in the Tasman region currently have an unassessed impact on flood routing. This is a knowledge gap which this study aims to address.

A condition assessment was conducted and found that the undocumented stopbanks had more woody vegetation growing in and around the stopbanks as well as more voids and surface damage than the Council stopbanks. The undocumented stopbanks were also grazed by livestock. Both woody vegetation and cattle grazing on stopbanks are recommended against in national (and international) guidelines (BoPRC, 2014; CIRIA, 2013).

To assess the effects of the undocumented stopbanks on inundation extent, a computational flood model was developed using the HEC-RAS hydraulic routing software, with the topographic data obtained from high resolution airborne LiDAR data. The flow boundary conditions were located at gauges (that recorded stage and estimated flow measurements) maintained by the Tasman District Council. There were several significant ungauged catchments within the model domain, flows from which were estimated using the Ministry of Works TM61 method (Ministry of Works, 1984). The model floodplain and channel roughness values were calibrated and validated against historic flood maps based on comparisons of inundation extent. In the analysis, the model was used in scenarios where the terrain was modified to represent the removal and maintenance of both Council and undocumented stopbanks.

The Council stopbanks were originally designed to protect against a 50-year flood event and maintain 0.60 m of freeboard. However, due to natural processes and gravel extraction, the stream bed has degraded significantly since the stopbanks were built.

Council stopbanks within the model were able to prevent inundation of the adjacent land where they were present, with no occurrences of overtopping. In a 100-year event simulation, the council stopbanks were able to maintain approximately a one metre of freeboard. Of the stopbanks assessed, the modelled Council stopbanks protected the largest area, which in a 5-year event was approximately 4.2 km^2.

In the modelled scenarios, the undocumented stopbanks all allowed inundation of the adjacent areas in all the return periods tested (the shortest return period was 5 years). A common trend was that their degraded condition allowed overtopping to occur, even in smaller events. Because of the lack of success in preventing inundation, scenarios were investigated to determine the effect of maintaining the undocumented stopbanks. In these scenarios, the inundation was significantly reduced. In one case, the stopbank contained more flow in the channel, causing a greater flooding extent downstream.

To quantify the impact of the undocumented stopbanks, Riskscape software was used, allowing the exposure, human displacement, damage states and reinstallment costs of buildings to be assessed. The human displacement and damage states both increased as the return period increased. The Council stopbanks stood to reduce the number of moderately damaged houses by 28% compared to when they were simulated to be removed. Most of these were located to the west of the Council stopbanks. This resulted in a net reduction in building reinstallment costs of $1.6M dollars in a 100-year event. In contrast, because the undocumented stopbanks did not significantly reduce the flood extent, they did not significantly reduce the number of moderately damaged buildings, human displacement or building reinstallment cost. However the undocumented stopbanks were originally designed to protect production land, which would likely be impacted by inundation, although this impact was not quantified. The maintained scenarios similarly did not significantly reduce the building damage states or reinstallment costs from the current stopbank scenarios. A key reason for this was that the land protected by the undocumented stopbanks was primarily production land with a low building density. The low number of buildings meant that despite the reduction in the inundation area, few buildings were exposed to flooding. However, within the inundation area more high-value orchards and vineyards are being developed, although the impacts to these were unable to be assessed.

Nationwide, other undocumented stopbanks may also be deteriorated as the undocumented stopbanks in the Waimea floodplain are. Without the improvements to flood protection systems if production land continues to be developed into higher value land the potential losses will likely increase in the future.

1. INTRODUCTION

1.1. Background

Floods are New Zealand's most frequent and damaging natural event because many towns and cities are located in flood-prone areas (Rouse, 2012). Within these flood-prone communities, flood levees (termed "stopbanks" in New Zealand) provide the primary physical means of preventing casualties and damage to property. It is estimated in New Zealand that there are currently more than 5000 km of stopbanks (Blake et al., 2018). These are managed by local and regional authorities as well as private ownership.

Local priorities drive Regional and District plans which control the management of stopbank structures as currently there is no New Zealand stopbank standard for construction. Some regional councils have however issued guidelines (BoPRC, 2014). Stopbanks throughout New Zealand vary in both quality and construction method because of this lack of a construction standard and also because of past decisions, community expectations, and different risk profiles (MfE, 2008). Nationally there are stopbanks not included in council assets. In many cases these stopbanks were built before formal consenting processes were introduced. As a result some stopbanks have not been exposed to a formal design and approval process. Consequently, the impact on flood routing of these stopbanks is currently unknown and could potential increase the flood impact by increasing the floodplain conveyance capacity.

Tasman District Council maintains 285 km of the district's rivers and 60 km of stopbanks that are owned, maintained and improved by the Council. In order to carry out its statutory role which includes mitigation of damage caused by floods, these rivers are maintained through a priority classification scheme. However, there are many more unclassified rivers, streams, and creeks in the Tasman District that are on privately owned land. Private flood protection structures on unclassified rivers are not typically administered by the Council. Documentation of privately-owned structures can be undertaken on an ad-hoc basis by Council or other organisations but records are typically incomplete.

Stopbanks for this study that are not catalogued by the Council and are not a Council responsibility to maintain are considered "undocumented stopbanks" with respect to formal Council management records.

The Tasman District Council acknowledges that it has several undocumented stopbanks within its jurisdiction. The Council has recognised that these structures have an impact that is often unacknowledged or poorly understood. The Council does not currently formally carry any documentation of their performance or physical characteristics (Griffith, 2018). Many of the stopbanks were constructed decades ago as small stopbanks by landowners to prevent shallower high-frequency floods in order to protect their property. Over time these stopbanks may have been raised by their owners to prevent larger floods. Today such earthworks at river margins would be governed by the Resource Management Act (1991):

> *"In achieving the purpose of this Act, all persons exercising functions and powers under it, in relation to managing the use, development, and protection of natural and physical resources, shall recognise and provide for the following matters of national importance:*
>
> *(a) the preservation of the natural character of the coastal environment (including the coastal marine area), wetlands, and lakes and rivers and their margins, and the protection of them from inappropriate subdivision, use, and development:" (MfE, 1991)*

However the stopbanks remain and their effect on flooding extent has not been assessed. In response to this, Tasman District Council has provided in-kind support for this project to better understand the impact of some of the more prominent undocumented stopbanks within their jurisdiction.

This study aimed to address a knowledge gap surrounding the unknowns associated with stopbank assets such as location, geometry and flood design capacity (Blake et al., 2018). This allowed the importance of undocumented stopbanks on the flood routing to be established. Outputs were used to determine the subsequent impact to people and property. This will help inform the Council and assist them to prioritise investment to minimise the future flood impact.

1.2. Scope and Objectives

The overarching aim of this study was to determine the impact of the undocumented stopbanks within the Waimea floodplain on flooding extent and subsequent damage. By addressing a knowledge gap regarding the condition and impact of undocumented stopbanks (Blake et al., 2018) generalisations may be applied regarding the condition and impact of stopbanks nationwide. It is hoped that this will help inform flood risk management decisions and aid prioritisation of investment to minimise flood risk. This is important as throughout New Zealand there are similar settings where the impacts of undocumented stopbanks on flood routing have not yet been assessed. Because of the uncertainty and potential adverse effects of undocumented stopbanks, there was clear benefit from researching their effect on flooding extent.

Features such as location, dimension, age, and flood capacity of undocumented stopbanks are unknown. Documenting these structures is essential to raise awareness of this important information and their general condition.

With this in mind, the objectives of the research presented in this thesis were:

1. To develop a geospatial data set containing information on location, geometry, and cover of the Council-owned and undocumented stopbanks within the Wai-iti and Wairoa catchments using a condition assessment.

2. To apply a general methodology (which could be applied to similar catchments) whereby a flood simulation model could be created, calibrated, and validated so that flood simulations could be undertaken.
3. Using these flood simulations, determine the impact that the undocumented stopbanks had on flooding extent and buildings to enable better informed investment decisions regarding flood mitigation strategies.

A range of scenarios were modelled and assessed as part of this study. Initial scenarios simulated and investigated the stopbanks in their current situation. Additional scenarios simulated the effects of removing the Council stopbanks and all the undocumented stopbanks. Several “removal” scenarios were simulated that removed the stopbanks individually to isolate their impacts on flood extent. This determined if the existence of the stopbanks were beneficial, detrimental, or redundant. “Maintained” scenarios predicted the effects of the undocumented stopbanks being uniform in height and geometry. These scenarios aimed to determine if it was beneficial in terms of flooding extent and building impact to increase the protection provided by the undocumented stopbanks.

1.3. Thesis Format

This thesis is arranged into eleven chapters. The present chapter covers the introduction to this study. This provides the background, objectives and motivation for the research.

Chapter 2 presents a literature review. This reviews the current global and national flood risk as well as protective measures taken to reduce the risk. This also includes a review of methods to develop flood maps and flood modelling software packages.

Chapter 3 introduces the relevant catchments, climate and stopbanks of the study area. This chapter also outlines the construction histories of the stopbanks and historical flooding in the study area.

Chapter 4 outlines the methods used in this thesis for gathering data on the condition of the stopbanks as well as methods used in developing and using the hydraulic model.

Chapter 5 presents the results of the condition assessment, detailing the cover, structures, roads and surface damage on the Council and undocumented stopbanks. This chapter also includes cross sections of the stopbanks. Stopbank condition guidelines are also presented to allow evaluation of the condition of the stopbanks.

Chapter 6 summaries the boundary conditions and sources of data used in developing the hydraulic model. These boundary conditions include flow inputs and topographic data.

Chapter 7 details the calibration of the Manning’s n values for the floodplain and channels in the model. The chapter also presents the results of the model validation against historic flood maps and gauge data.

Chapter 8 uses the model validated in Chapter 7 to simulate scenarios that investigate the current impacts that the Council and undocumented stopbanks have on flood extent with return periods ranging from 5 to 100 years. Additional scenarios also assess the impact of removing and maintaining the stopbanks on flooding extent.

Chapter 9 uses the flood inundation depths, velocities and duration with Riskscape software to assess the impacts of the Council and undocumented stopbanks have on buildings. The assessment measures the building damages, human displacement and reinstallment cost. The results are compared to highlight the potential benefits and consequences of removing and maintained the Council and undocumented stopbanks.

Chapter 10 outlines the limitations of this study with regard to the hydraulic model inputs, model boundary conditions and the building impact assessment.

Chapter 11 presents a general summary of the study, a discussion and recommendations for further research.

2. LITERATURE REVIEW

2.1. International Flood Risk and Protective Measures

Globally, floods are one of the most serious climate-related hazards. It is estimated that floods constitute 43 % of the total number of natural disasters that affect 2.3 billion people causing a total damage of approximately USD 662 billion between 1995 and 2015 (UNISDR, 2015). Floods have become more frequent and larger in magnitude in recent decades (UNISDR, 2015). Urbanisation and the sealing of the ground surface has significantly increased water runoff in many areas (Kundzewicz et al., 2019). People are however increasingly inhabiting flood-prone areas in many countries (Hardoy, Mitlin, & Satterthwaite, 2001; Salami, von Meding, & Giggins, 2017). Where flood protection systems are present they aim to reduce the losses experienced. In areas of infrequent flooding, residents can become less aware of the threat floods pose to them despite the flood protection systems.(CIRIA et al., 2013). Residents in these cases may be unprepared for floods and unsure of proper actions to take. This can result in communities suffering more serious damage when a flood does occur (Ericksen, 1986).

Stopbanks, otherwise known as levees, dikes or embankments, are a critical aspect of flood management plans globally. Most countries utilise stopbanks alongside rivers and in coastal areas as part of their protection systems. Due to their extensive length, maintenance of these stopbanks is a major undertaking for the flood management authorities. It is estimated that there are several hundred thousand kilometres of stopbanks in Europe and the USA alone (ICOLD, 2018).

Floods can cause critical flood protection structures to fail leading to loss of life and the devastation of large areas of land. Despite stopbanks' critical importance in reducing flood risk, interest and investment in them has tended to be lower than with other water retaining infrastructure such as dams (CIRIA et al., 2013). In many countries, stopbanks have lacked the legal and technical framework necessary to promote an appropriate level of performance and their development has suffered as a result (CIRIA et al., 2013).

National design guidelines for stopbanks exist in some areas such as British Columbia (*'Dike Design And Construction Guide: Best Management Practices For British Columbia'*, (Golder, 2003), and the USA (*'Design and Construction of Levees'*, (USACE, 2000). These documents aim to guide site investigation, design, construction and maintenance of stopbanks. Worldwide, the design and construction techniques vary and many of the techniques used do not necessarily take full advantage of experience in other countries. Several governments have recognised the benefit in collaboration and have sponsored the production of a single reference that captures current best practise for management and design of stopbanks. The result of this was the *'International Levee Handbook'* created by a collaboration between the USA, France and the UK, with additional support from Ireland, the Netherlands and Germany (CIRIA et al., 2013). While it is not a legal document, it represents current best practice in stopbank design and construction.

2.2. Flood Risk in New Zealand

Floods are New Zealand's most frequent and damaging natural event because many New Zealand towns and cities are located in flood-prone areas (Rouse, 2012). On average a major freshwater flood occurs every eight months with the frequency and intensity likely to increase due to climate change (Brennan, 2015). The most severe flooding historically occurs from events with high intensities and long durations. A study showed that within New Zealand, the highest intensities of rainfall occurred in Northland, the West Coast and Tasman (Tomlinson, 1978). In 2018, New Zealand was affected by two ex-cyclones, Fehi in early February and Gita later in the month. Both of these events had large amounts of rainfall that caused significant flooding. During Gita, 148 mm of rain fell over a 14 hour period. This represented 173% of the average February rainfall for the nearby Motueka-Riwaka area. These events made it the wettest February in Nelson in the 156-year record (NIWA, 2018a). In December of that year, a thunderstorm caused rainfall that led to slips that closed State Highways 4 and 25. The event also resulted in five adults needing to be rescued from a vehicle that was swept along by floodwaters (NIWA, 2019a). It is estimated that there are almost 700,000 people and 135 billion dollars' worth of buildings exposed to river flooding in the event of extreme weather events in New Zealand (NIWA, 2019b).

The net effect of climate change is that it will likely lead to an increased risk from floods in New Zealand (NIWA, 2016). For each degree Celsius of warming, saturated air contains 7% more water vapour (Coumou & Rahmstorf, 2012) which may form rain if conditions are right. Increased moisture content also provides more latent energy to drive storms. The increased flood risk stems from climate change causing an increased frequency of heavy rainfall events that could increase the flood inundation hazard, along with sea level rise (NIWA, 2019b). Using NIWAs' national hydrological model and six downscaled Global Climate Models, NIWA (2016) predicted the Mean Annual Flood (MAF) across New Zealand for four Representative Concentration Pathways (RCP 2.6, RCP 4.5, RCP 6.0 and RCP 8.5) over the 2036 to 2056 and 2086 to 2099 periods. This process allowed the difference in current and future predictions of the MAF to be compared. Generally the simulations indicated the MAF would increase in the country's agricultural areas and that the MAF increased more in the more extreme RCPs (NIWA, 2016). Overall climate change is very likely to increase the frequency of flood events and flood risk nationwide (MfE, 2008).

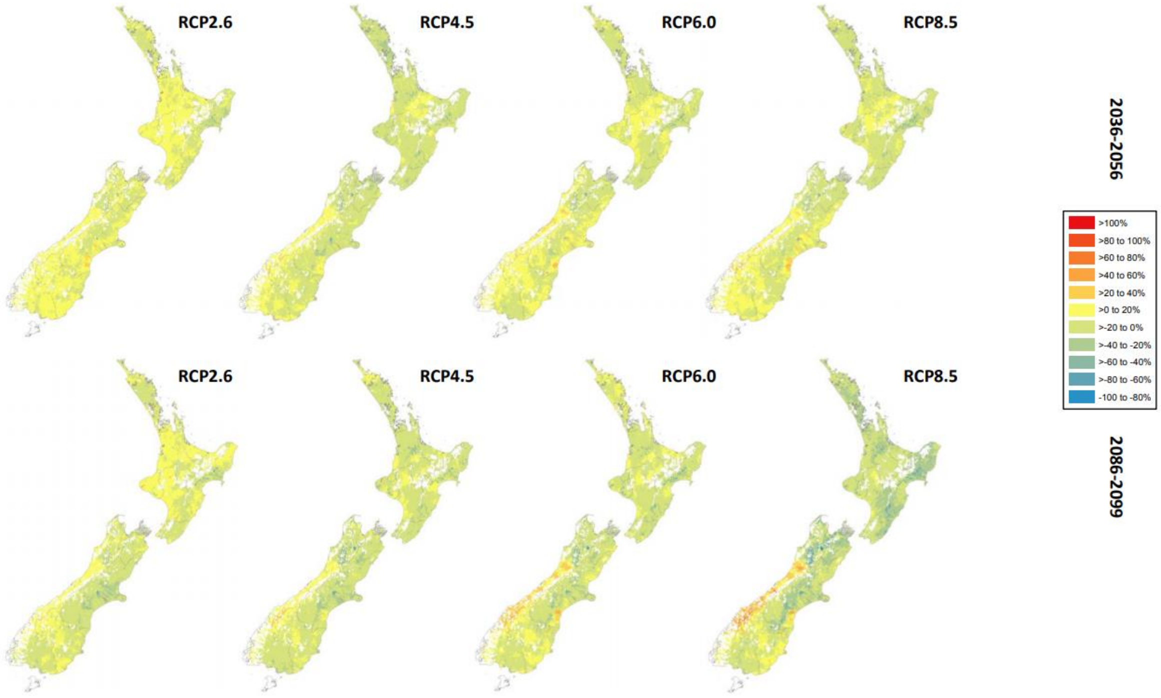


Figure 2.1. Median percentage changes in mean annual flood flow (NIWA, 2016)

To account for potential increases in rainfalls under climate change it is now best practise to increase rainfall intensities when modelling floods (MfE, 2010). Evidence suggests that in the future there will be an increase number of large scale flooding events and appropriate measures should be taken to prepare for them (Knight & Shamseldin, 2006). However, the current levels of flood protection are unlikely to be sufficient under the effects of climate change unless the current standards are reassessed and strategies to manage risk are put in place. Despite this, development of floodplains is continuing to occur (MfE, 2008).

2.3. Flood Losses in New Zealand

The cost of flood losses is generally separated into three major categories: casualties, social disruption and property.

In spite of the frequency of flooding within New Zealand few lives are lost. However there is no comprehensive natural hazard database to determine the annual deaths due to floods. Between 1990 and 2012, a total of 167,555 deaths were attributed to flooding worldwide (CRED, 2019), an average of 7,616 lives per year.

Social disruption relates to the termination and rescheduling of activities due to flooding. These are also often called indirect losses. These costs are not usually measured or well documented (Ericksen, 1986). It is reasonable however to assume that the more severe an event, the more the social disruption will increase. The losses to income and production can be conservatively lowly estimated as equal 15 percent of the direct losses (Ericksen, 1986).

The direct cost of property and contents damage to landowners can be minimized by insurance. Insurance externalises the cost of flood disaster in that those unaffected subsidise the true cost of floods. This method does not provide a deterrent for building in a known flood-prone area (Ericksen, 1986). The Insurance Council of New Zealand (ICNZ) has acknowledged this and has stated that insurance needs to provide a stronger signal of risk levels to the public whilst also remaining affordable to everyone (ICNZ, 2014).

There is a lack of substantive New Zealand flood loss analyses (Ericksen, 1986). Previous research into the distribution of flood losses combines the effects of the flooding in Nelson and New Plymouth in 1970 and 1971. The events are estimated to have had a total flood loss cost of 102.5M dollars (2004 NZD) (NZIER, 2004). The event in Nelson was assessed at the time as a 50-year return period, and the flood in New Plymouth had a 100-year return period. A breakdown of these costs is shown below in Table 1. This table has been adapted from (NZIER, 2004)

Table 2.1. Flood losses for Nelson and New Plymouth 1970 and 1971 (NZIER, 2004)

DIRECT LOSSES	**NZD Millions, 2004 dollars**
Central Government works and services	
• Roading	28.0
• Railways	3.7
• Bulk Power Supply	0.7
• Flood control and drainage works	11.4
• Sub-total	43.8
Local government	
• Roading	4.0
• Flood control and drainage	3.9
• Water, sewage and telecommunications	2.7
• **Sub-total**	10.6
Private Sector	
• Farmland	0.5
• Disaster fund pay-outs	5.7
• Insurance industry pay-outs	17.1
• Uninsured property	11.4
• Sub-total	34.7
• **Total direct assets**	**89.1**
INDIRECT LOSSES	
• **Income and production**	13.3
TOTAL FLOOD LOSSES	**102.5**

In 2017, the remnant of two extratropical cyclones, Cook and Debbie, passed over New Zealand causing flooding in many areas along the east coast of the North Island. During the flood, a section of stopbank at Edgecumbe failed. The losses that were insured against extreme weather events (predominantly flooding) in 2017 exceeded $200M (ICNZ, 2019).

2.4. New Zealand Protective Measures Against Flooding

In New Zealand, there are approximately 100 flood-prone communities where the primary physical means of protection from inundation is in the form of stopbanks. Stopbanks have been selected as the most practical solution against floods for various reasons including that they protect against floods whilst allowing ongoing development on the floodplain. This can increase the economic revenue of the area making it an appealing choice for local governments (Ericksen, 1986). It is estimated that there are currently more than 5000 km of stopbanks across New Zealand (Blake et al., 2018). These are managed by local and regional authorities as well as some in private ownership and management. However, stopbanks can lead to catastrophic flood losses through a positive feedback loop. Stopbanks encourage the intensification of floodplains, as areas are perceived as protected. The intensification leads to an increase in the value of the land and assets at risk. As a result, there is a demand for higher stopbanks and more protection. This in turn encourages and further intensification. A significant flood will inevitably occur and cause breaching and lead to catastrophic flooding as stopbanks cannot provide absolute protection. This cycle of development, stopbank raising, to more development has happened across New Zealand as well as many other countries (Ericksen, 1986).

The maintenance of stopbank structures is generally covered by the Local Government Act (2002) under which local authorities play a broad role in meeting the current and future needs of their communities for good-quality local infrastructure (DIA, 2002). The construction and maintenance activities on the stopbanks are governed by the Resource Management Act (1991), under which the natural character of rivers and their margins shall be preserved and provided for (MfE, 1991). The responsibility for the management of stopbanks is directed by Regional and District Plans which are driven by local priorities. Currently, there is no nationwide standard for the construction of stopbanks however there are guidelines for their design and construction issued by some regional councils such as the Bay of Plenty's *'Stopbank Design and Construction Guidelines'* (BoPRC, 2014). A review in 2014 concluded that flood risk management knowledge and expertise will continue to develop in an ad-hoc manner. The report also stated that this will likely result in unnecessary duplication of effort occurring due to the lack of national leadership (Whitehouse, 2014).

In general, there are many similarities between New Zealand stopbank design guidelines from the Bay of Plenty and international guidelines in the International Levee Handbook (CIRIA et al., 2013). Both documents agree that the primary function of a stopbank is to provide protection in the flood event. The documents also agree on best practice for maintenance and the failure mechanisms of stopbanks. Both documents acknowledge the same measures taken to prevent such failure. However, the International Levee Handbook is more comprehensive in its analysis and addresses a broader range of topics concerning stopbanks. These similarities are because the Bay of Plenty guidelines draw much of its knowledge from the International Levee Handbook which is acknowledged in the foreword.

The construction of New Zealand's stopbanks began in the late 1800s before the development of modern engineering flood protection practices (Ericksen, 1986). The quality and construction methods of stopbanks vary greatly throughout the country depending of past decisions, community expectations, resources available, and risk profile of each area (MfE, 2008). Despite council efforts, in some regions of New Zealand there are a large number of privately-owned or unconsented stopbank structures, here on referred to as "undocumented" stopbanks. Because undocumented stopbanks are not part of a council's assets they have not been subjected to the rigours of the formal design and consenting processes, and may not have had proper maintenance. As a result of this, undocumented stopbanks may pose a significant but unassessed impact on flood routing. While the stopbanks may be reducing the risk as intended, it is possible that they could be raising the flood risk to downstream communities.

There is currently a knowledge gap regarding the importance of these undocumented and privately-owned structures to the flood routing (Blake et al., 2018). These structures could unknowingly and unnecessarily be increasing the flood risk in certain areas. Assessing the impacts of this would allow the determination of the subsequent risk to people and property. This will inform the Council of any benefits or detriments the stopbanks have regarding impacts and assist them to prioritise investment to minimise flood risk. Understanding the influence undocumented stopbanks have on flood extents would also inform the Council of the impact of these stopbanks as part of their due diligence processes.

2.5. Tasman Flood Risk

Tasman has been highlighted as one region that is prone to flooding (Ericksen, 1986). The wider Nelson area was expected to experience 150 mm of rainfall in one day, once every twenty years in 1986 (Ericksen, 1986). Because of this risk, Tasman is prone to severe flooding that has the potential to cause significant damage (NZIER, 2004). When flooding occurs, it represents a high cost for the Tasman region and a reduction in these losses would be of benefit to the community. In 2014, the insured losses from the Nelson-Tasman floods totalled $2.7M (ICNZ, 2019).

In order to manage flood losses, the Tasman District Council (TDC) has as part of its assets, 60 km of stopbanks that are owned, maintained and improved by the Council. However, there are many rivers where private undocumented stopbanks exist.

2.6. Current Stopbank Maintenance Best Practice

Internationally the most detailed reference of stopbank maintenance best practice is the International Levee Handbook (CIRIA et al., 2013) because it was created collaboratively by the many of world's authorities on stopbanks. Within New Zealand, the Bay of Plenty's (BoP) 'Stopbank Design And Construction Guidelines' (BoPRC, 2014) is representative of New Zealand's best practices.

The BoP guidelines indicate that the maintenance of a stopbank is critical to its long-term performance. Maintenance should be proactive in identifying and addressing issues with stopbanks: failure to do this may raise the flood risk to the surrounding inhabitants and property. Some of the critical aspects of the stopbank maintenance are condition assessments, vegetation management, and penetrations.

The International Levee Handbook and BoP guidelines both agree that regular condition assessments should be carried out at least annually. Assessments should also be undertaken during and after significant flood events. These inspections are important as they allow damage to be documented and provide the foundation for updating the maintenance program. This is critical to ensure ongoing stopbank maintenance to meet the required level of protection.

The BoP guidelines and International Levee Handbook recommend that trees growing on a stopbank should be removed as soon as possible. Within the BoP region, permission is required from the Council before planting any vegetation on or within 12 m of the downstream slope or anywhere between a river and stopbank. The reasons for this, as stated in the International Handbook, are that trees can damage the stopbanks in a many ways such as overturning, root penetration, and discouraging grass growth which may lead to increased surface erosion. Although it is internationally acknowledged that woody vegetation should be removed from stopbanks, there is no consensus on the extent of roots that should be removed. This is in part because the agencies disagree about the risk roots pose to stopbank deterioration, with several differing approaches specified depending on the agency (CIRIA et al., 2013):

- German standards require the complete removal of the root system and reconstruction of the affected area.
- Dutch guidelines recommend that the roots should be removed as much as possible, and replaced with engineering fill. In practice, this does allow the stump and roots to be left in place after the tree is cut flush to the ground.
- USACE guides recommend the removal of the stump, root ball, and root greater than 13 mm in diameter. The voids should then be filled with material matching the original soil specifications.

The BoP guidelines state that "trees including all root systems should be removed to ensure that no seepage paths remain." (BoPRC, 2014)

The recommended surface cover for stopbanks both domestically and internationally is trimmed grass. It is cost-effective, resilient, and prevents external erosion. Good vegetation management allows stopbanks to be visually inspected. This is achieved by mowing or trimming the grass cover. The BoP guidelines recommend that livestock should not be used for this purpose as they can destroy the grass cover and expose the stopbank surface to erosion. Cattle can also cause settlement and rutting, resulting in loss of freeboard and the stopbank being unable to meet the desired performance standard.

Penetrations such as roots, powerlines, pipelines, cables and other structures are a potential source of damage to stopbank integrity. The BoP guidelines acknowledge that stopbanks must occasionally be penetrated for installation or maintenance of services. This should be done so as not to create a weakness in the stopbank. Within the Bay of Plenty region, permission from the Council must be sought to:

- Plant any vegetation on a stopbank or within 12 m of the landward side of any stopbank or between the watercourse and stopbank.
- Construct any structure in a stopbank or within 12 m of the landward side of any stopbank or between the watercourse and stopbank.
- Carry out any excavation between the watercourse and stopbank.
- Carry out any excavation including for building foundation, within 20 m of any stopbank.

The potential consequences of penetrations are cited in the International Handbook as a reduced seepage path that can lead to piping and internal erosion causing the stopbank to fail.

2.7. Flood Mapping and Modelling

There are many benefits to society for developing accurate flood maps. Within the US, it is estimated that flood maps are used 30 million times annually by government agencies (FEMA, 2009). Flood maps can be used to identify areas at risk of flooding, this useful to ensure adequate preparation by Civil Defence and other agencies to prioritise response efforts. Flood maps can also be used to communicate flood risk and aid in raising public awareness of flooding extent (FEMA, 2009). Flood mapping is also a vital component of proper land use management in flood-prone areas as it allows increased accuracy in classification of areas at risk. Risk classifications can govern the restrictions placed upon land which form the foundation of land use management (FEMA, 2009). In areas without flood maps, it is difficult to communicate flood risk to community officials or citizens. Internationally, the sale of flood insurance is not mandated in areas outside of flood maps which affects the effectiveness of construction and insurance regulations in relation to flooding (ASFPM, 2013).

Flood maps are an essential tool in avoiding or minimizing the damage to property and loss of life caused by floods. Within New Zealand, flood map accessibility has recently improved with public flood hazard maps available for thirteen regional councils and unitary authorities (NIWA, 2019b). Flood maps can be created using a range of methods including ground surveying, aerial photography, remote sensing and hydraulic modelling.

Ground surveying, aerial photography and remote sensing have in many cases been used to create historical flood maps. Historical flood maps serve many purposes such as a data source from which numerical models can be calibrated. Historical maps can also be compared to future flood maps to assess how flooding extents have increased and decreased spatially over time (Brakenridge, Andersona, Nghiemb, Caquard, & Shabaneh, 2003). Ground surveying and aerial photography need to be completed as quickly as possible after an event to capture the flood footprint to assess the magnitude of losses (Aggarwal, 2016).

Remote sensing systems further develops the use of traditional aerial photography through the use of satellites as well as aircrafts. Remote sensing can provide much of the information needed (such as land cover type, topography, and inundation extent) to map flood extents of a flood as well as assess the resulting damage (Klemas, 2015). Conventional hydrological monitoring systems are limited in their ability to forecast floods and cannot measure flood extent directly. The cost of maintaining gauging stations is costly and often a limiting factor in their use. Although only gauging stations can give accurate water depths. Issues also arise using gauging stations where a river flows through multiple countries but the information is not communicated downstream (OAS, 1991). Remote sensing addresses these issues. It has also been demonstrated that remote sensing can be used as a cost-effective method for hydrologic prediction in poorly gauged or even ungauged basins (Khan et al., 2011).

Hydraulic models are computational models that attempt to replicate fluid motion through solving governing equations. These models are able to be manipulated to investigate the impact of changes in terrain elevations, boundary conditions and initial conditions to account for installation or removal or destruction of hydraulic features. For example, certain return period floods may be modelled or the flows may be increased to account for the increased runoff due to climate change (SKM, 2013). Other scenarios that can be simulated include the inundation extent from a dam breach (Knight & Shamseldin, 2006) or the increase in inundation extent due to urbanisation (Park & Lee, 2019). Hydraulic models require parameters such as terrain, flow inputs and roughness values to be defined. Accurate topographical and cross-section data are critical to the accuracy of the final flood maps produced by hydraulic modelling (FEMA, 2009)

2.8. Flood Modelling Software

Using computational hydraulic models to simulate surface runoff is a complex task. However, because these models are able to simulate user-defined scenarios they are often heavily relied on in catchment flood management plans and river basin management (Knight & Shamseldin, 2006). These models can be grouped by their dimensionally (1-D, 2-D, and 3-D).

The simplest representation of flow is 1-D. These models are often used where more detailed solutions are unnecessary or where flow is essentially 1-D such as in a pipe. Because 1-D models assume flow travels perpendicular to cross-sections, they are limited in their ability to represent urban floods (El Kadi Abderrezzak, Paquier, & Mignot, 2009). Although there is no standard for what tool is used to generate flood maps in New Zealand it is likely that 1-D models are the most common flood model used by regional councils and territorial authorities. This may be because they are cheaper and easier to run without detailed topographic data required in more complex models (Rouse, 2012). In contrast, 3-D models are not commonly used in flood modelling due to the longer model run times, however, unlike other models, they can represent hydraulically complex inlets (Swift, 2014). Usually water depths are shallow compared to the flooding extents and therefore it is generally acceptable to use 2-D models to represent floods.

2-D models are also commonly used in floodplain modelling however they require longer times to develop and simulate floods than the 1-D models (Gharbi, Soualmia, Dartus, & Masbernat, 2016). Comparisons of 1-D and 2-D models have shown that 2-D models can provide more realistic representations of flooding in topographically complex floodplains (Cook & Merwade, 2009; Gharbi et al., 2016; Tayefi, Lane, Hardy, & Yu, 2007). Within New Zealand, 2-D models are more likely to be used by better-resourced councils or in areas with a large potential flood risk (Rouse, 2012).

There are several important aspects of a modelling software such as the governing equations, numerical scheme, methods of time discretisation, and method of spatial discretisation. Many 2-D hydraulic models solve the shallow water equations (or simplifications of them). These equations have no analytical solution therefore numerical schemes are used to determine approximations. Depending on the numerical discretisation, models can be classified as either finite element, finite volume or finite difference. Finite volume schemes have become the most widely used because they guarantee conservation of mass and momentum and are geometrically flexible and is conceptually easy to understand (Alcrudo, 2004). Models can also be classified based on their method of time discretisation as either implicit or explicit. Implicit models solve the governing equations across the entire domain before proceeding to the next time step. Explicit models solve for each cell independently of the domain for each time step. The methods used for spatial discretisation can also be used to classify models, the most common methods are: structured (rectangular) mesh, unstructured mesh, and flexible mesh. Commonly used 2-D software packages are outlined in Table 2.2 (Teng et al., 2017).

Table 2.2. Common 2-D hydraulic models and their technical aspects

Model Name	Developer	Governing Equations	Numerical Scheme	Time Discretisation	Spatial Discretisation	Status
Flood Modeller Pro	CH2M Hill	Shallow water equations	Finite difference	Implicit	Structured mesh	Commercial
HEC-RAS 2D	USACE	Shallow water equations	Finite volume	Implicit	Flexible mesh	Free
LISFLOOD-FP	University of Bristol	Shallow water equations	Finite difference	Explicit	Structured mesh	Research
MIKE 21	DHI	Shallow water equations	Finite volume	Explicit	Flexible mesh	Commercial
TELEMAC	Electricite de France	Shallow water equations	Finite element/finite volume	Implicit/explicit	Unstructured mesh	Open source
TUFLOW FV	BMT WBM	Shallow water equations	Finite volume	Explicit	Flexible mesh	Commercial
TUFLOW HPC	BMT WBM	Shallow water equations	Finite volume	Explicit	Structured mesh	Commercial

These packages (and others) were compared in several scenarios developed by the British Environmental Agency in a report (EA, 2013) that aimed to provide evidence that modelling packages were capable of adequately predicting floods (USACE, 2018). These scenarios included valley flooding, dam-break flooding and rainfall surcharge. The report concluded that generally the packages produced comparable predictions of depth and velocity across the full range of tests. There were two exceptions where the packages should not be used. Firstly, where the domain was greater than 1000 km^2 as this led to long simulation runtimes. Secondly, where high detail was needed at transitions between subcritical and supercritical flow, such as at dam or embankment breaches. Several similar studies have found that many software packages produce reasonable and consistent results for a range of scenarios such as floodplain inundation and urban flooding, (Horritt & Bates, 2001; Hunter et al., 2008; Paudel, Roman, & Prichard, 2016).

Since the release of the report by the EA (2013) software packages have continued to be developed and improved. Updated versions of these software packages have been benchmarked against the original scenarios and have proven to be consistent or superior to the packages presented in the original report (BMT, 2017). The software used for flood modelling in this study was HEC-RAS 2D as it was freely available and has been shown to produce comparable predictions to other commercial software packages.

2.9. Summary

Floods are amongst the worst natural disasters internationally and result in the assessed losses of tens of billions of dollars in damages and thousands of lives lost each year. Stopbanks aim to reduce this impact. Within New Zealand floods are also the most frequent and damaging natural hazard. Serious flooding causes casualties, property damage and social disruption. Large scale flood events represent a high cost to communities and it would be of benefit if this cost could be reduced.

Nationally there are over 100 flood-prone communities where stopbanks provide the primary means of physical protection in many cases. It is estimated that in New Zealand, there are currently more than 5000 km of stopbanks. A review concluded flood risk management knowledge and expertise is likely to continue to develop in an ad-hoc manner with unnecessary duplication of effort occurring without the establishment national leadership.

The construction of New Zealand’s stopbanks began in the late 1800s and despite council efforts, some regions in New Zealand contain a large number of privately-owned or unconsented stopbanks. These stopbanks have generally not been subjected to the rigours of formal design, the consenting processes, or proper maintenance. This has resulted in some undocumented stopbanks having an unknown effect on flood impact. Knowledge of this information would inform councils and allow them to prioritise investment to minimise the flood risk. TDC is one council that acknowledges it has several undocumented stopbanks within its jurisdiction.

There are many wide-ranging benefits for developing accurate flood maps such as identifying areas at risk for emergency responders, communicating flood risk and in classification of flood-prone areas as part of proper land use management. Within New Zealand, flood map accessibility has recently improved with flood hazard maps publically available for thirteen regional councils and unitary authorities.

Historical flood maps have in many cases been created using ground surveying and aerial photography. As well as the previously outlined benefits, historical flood maps can be used to calibrate computation models. Computation models attempt to replicate fluid motion by solving governing equations. An advantage of hydraulic modelling is that models can be modified to investigate the impacts of alterations in terrain and boundary conditions. Because of this hydraulic models are often relied upon in flood management plans. 2-D hydraulic models are the most commonly used models. These can be categorised based on their numerical schemes, time discretisation, and spatial discretisation. It has been shown by multiple studies that these software packages generally produce comparable predictions of depth and velocity.

3. STUDY SITE

3.1. River Catchments

The Waimea floodplain drains an area of approximately 720 km^2 at the north of the South Island of New Zealand. Within this area, there are two main catchments, the Wairoa and the Wai-iti. The larger Wairoa Catchment drains approximately 460 km^2 (SKM,2013). A broad overview of the relevant catchments and land use is shown in Figure 3.1.

(a) (b)

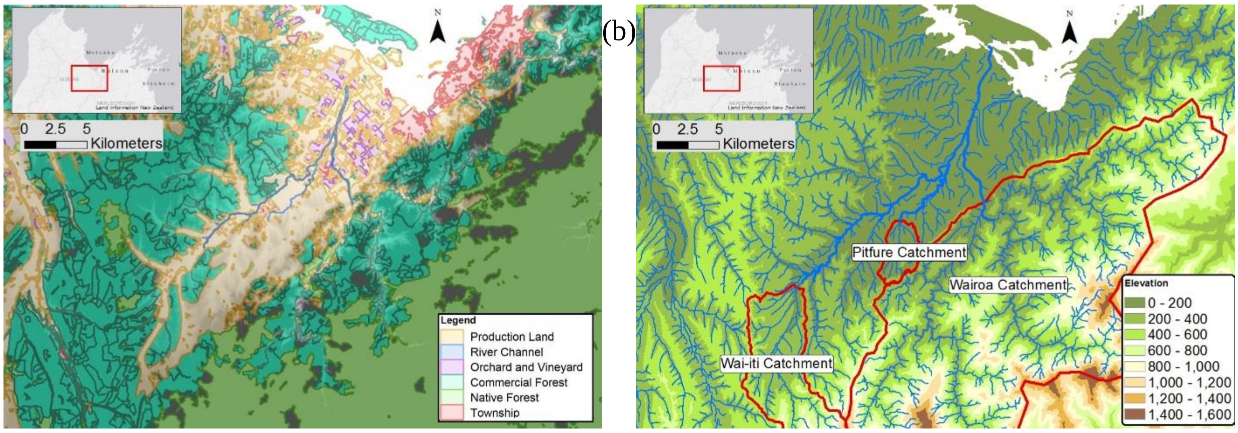


Figure 3.1. Waimea floodplain overview of: (a) land use catchments (b) relevant catchments

The upstream areas of the Wairoa catchment are steep and dominated by indigenous forest made up of tall forest canopy species. This area is protected by the Department of Conservation (DOC) as the land is part of the Mount Richmond Forest Park. Downstream the land is mostly pine forest that is used for commercial forestry purposes. After following through this, the Wairoa River flows through a gorge and enters the Waimea floodplain at an elevation of 30 m. The Wairoa flows for approximately 6 km until, north of Brightwater, the Wairoa joins with the Wai-iti River.

The Wai-iti River is the smaller of the two rivers that form the Waimea. The Wai-iti River drains a catchment of 290 km^2. The steep mountainous area of the upper Wai-iti is used for commercial forestry. The Wai-iti River drops down a steep valley to enter the floodplain at an elevation of 300 m. Once the Wai-iti has exited the valley it is joined by many smaller tributaries. It then flows for 20 km through production land before joining the Wairoa to form the Waimea River.

After the confluence the Waimea River flows for 10 km through a mixture of production land, grassland and orchards before reaching an outlet to the sea. The outlet to the sea is heavily tidal dependent with backwater effects up to 3 km up the Waimea River. At the outlet, the Waimea River is immediately faced with Rabbit Island and an estuary. This outlet, because of these factors, is a hydraulically complex region.

The Pitfure Stream is a tributary of the Wai-iti River. Despite the Pitfure Catchment making up only 5% of the area of the Wai-iti Catchment (11.5 km^2) and having a tendency to run dry for long periods, its location near Wakefield makes it play an important role in flooding (Zemansky, Hong, Rose, Song, & Thomas, 2012). The stream flows through mostly production land for nearly 5 km before flowing past Wakefield Township. The streamflow in some locations is less than 100 m from residential buildings. An earlier report highlighted that the Pitfure appears to cause much more regular flooding problems than the Wai-iti and Wairoa Rivers, and has the potential to cause significant damage in an extreme event (SKM, 2013). The Pitfure, after passing Wakefield, continues for 7.5 km through production land to merge with the Wai-iti River.

The land in the Waimea floodplain is primarily used for horticultural (mainly apples and kiwifruit) and agricultural purposes. The main source of irrigation comes from groundwater. The groundwater also acts as the main source of water for the surrounding population of approximately 18,000 inhabitants. However, a significant amount of water is taken from the Waimea River to irrigate the intensively cropped land (Walrond, 2015).

A population of 15,000 resides within Richmond. Outside of this, there are two population centres along the Wai-iti and Wairoa Rivers. Brightwater and Wakefield both have populations of 2000 residents. These town centres are mainly residential and support agricultural services, although there are some light industrial areas. A map of the study area is shown in Figure 3.2.

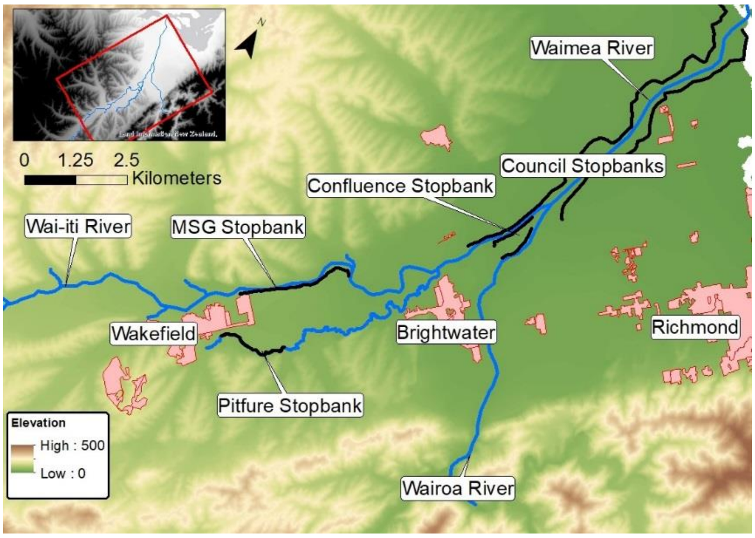


Figure 3.2. Key urban areas and stopbanks of the study area

3.2. Geology

The type of soil and rock in an area can affect the amount of runoff during a flood event as some soils have a higher infiltration capacity than others. Different soil and rock coverings also have different roughness values that influence the speed at which runoff flows over them.

The soil within the Wairoa was most commonly either steepland soil or loams. Within the upper areas of the Wai-iti catchment and its tributaries the soil was primarily hill soils. Within the floodplain, the soil varied between, sand, silt, clay, and gravelly loams (Manaaki Whenua, 2019) .

3.3. Climate

The Tasman district, in general, is the first to be exposed to weather systems moving for into the South Island from the north but is relativity sheltered from systems travelling from the south. The rainfall in the region is fairly evenly distributed temporally, with the driest months in late summer and the wettest in winter/spring. Specifically, the Waimea floodplain is one of the driest areas in the region because of shelter from the west and south rain systems (Macara, 2016). In the floodplain, the average annual rainfall is 1500 mm, but this varies greatly spatially due to the position and form of the surrounding hills (SKM, 2013).

The temperature in the region is mild due to the close proximity to the sea which results in a lack of extreme highs and lows. However, the region and Nelson in particular, are well known for the number of sunshine hours received, which are among the highest in the country (Macara, 2016).

3.4. Construction Histories

Within the study area there were four main stopbanks that were examined. The undocumented stopbanks are the Main Spring Grove, Pitfure and Confluence stopbanks. The TDC maintains the length of the stopbanks along the Waimea River, in this study, these stopbanks are referred to as the Council stopbanks. The locations of these stopbank is shown in Figure 3.2.

3.4.1. Council Stopbanks Setting

The Waimea River Park extends from upstream of the confluence to below the Brightwater Bridge. The park covers the land alongside the river, shown in Figure 3.3, (TDC, 2010). The TDC holds the land for this park in two titles, the larger parcel of land (3.92 km^2) was acquired for river control purposes. The smaller land parcel (0.017 km^2) is situated at the Appleby Bridge Recreation Reserve and has been classified under the Reserves Act by the TDC (TDC, 2010).

The purpose of the Reserves Act 1977 is to protect land, identify and protect natural and cultural values and provide for public *access*. In this case, the smaller land parcel is classified under the Reserves Act. Local Purpose Reserves are classified "*for the purpose of providing and retaining areas for such local purpose or purposes as are specified*" (Department of Conservation, 1977). The primary purpose of management, in this case is soil conservation.

The larger parcel that covers most of the park is not classified under the Reserves Act. Because of this, the policies under the Act are not enforceable under the act and that the policies have the same weighting as TDC policies.

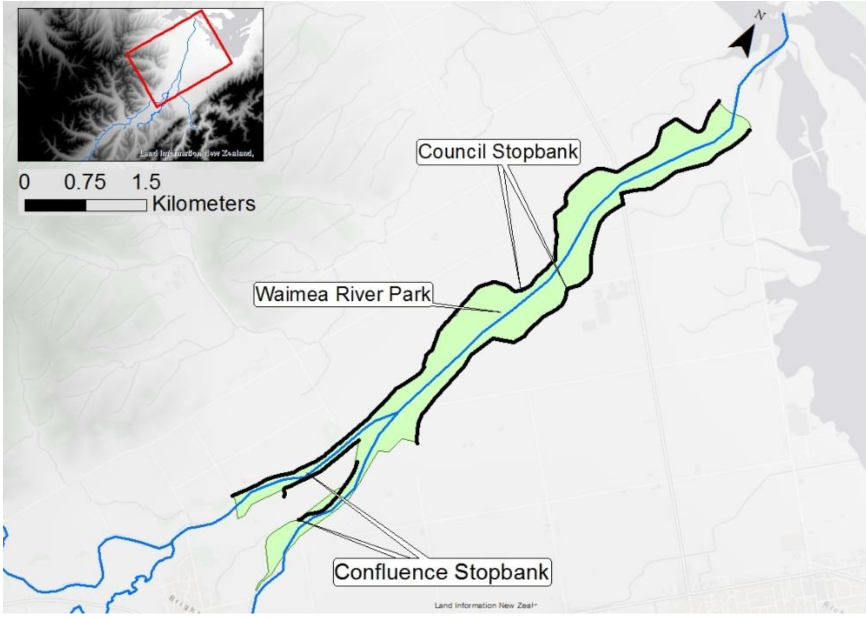


Figure 3.3. The extent of Waimea River Park

In order to control the flooding, in the early 1960s works were actioned to construct stopbanks. These Council owned stopbanks run continually along the length of the park. The stopbanks are set between 250 to 1150 m apart. The stopbanks were originally designed to protect against a one in the fifty-year event (2% AEP), with 0.6 m of freeboard. As the stopbanks are part of the TDC assets and an important piece of infrastructure, the Council maintains the stopbank, regularly mowing the stopbank so that it is free of vegetation. As a result of the management of the Waimea River Park the river has been artificially straightened. Since the stopbanks have been constructed, the use of land upstream has been more intensely forested commercially. This intensification includes the removal of trees, this has likely increased the nutrient flow as well as the frequency and nature of flooding for the catchment.

Within the Waimea River Park, the river's edge has been extensively altered over the years through engineering works such as gravel extraction and stopbank construction. Previously, along the East bank, there have been three gravel processing plants with their associated stockpiles. In the past, the Waimea was an important source of gravel for the construction industry. Originally the extraction rate from the river bed was greater than the rate being deposited naturally. Over time this lowered the river bed and water levels within the Waimea aquifers. As a result, the extraction rate in 2008 was reduced to 3000 m^3 and ended altogether in 2009. As of 2010, gravel was still being processed from the river banks (TDC, 2010).

The gravel processing plants contribute most of the income ($200,000) made from land leases within the park. This money is used by the Rivers account to fund works on the river banks. The Rivers account exists to fund the maintenance and development of the rivers and is funded primarily through targeted rates. The leases range from five to 20 years. Although gravel processing is the dominant commercial income of the Park, the majority of leased land is used for livestock grazing. Grazing land between the stopbanks is used to control vegetation and provide an income to the Rivers account to manage the works within the park. The primary object of the Waimea River Park as stated in the 2010 River management plan is (TDC, 2010):

> *"Manage the riverbed and berm lands within the park to protect surrounding lands from flood flows of the Waimea River, to assist in maintaining the Waimea Plain aquifer and to maintain water quality."*

This objective is aided by the maintenance of the stopbanks that is funded by the income from leased land. This objective is the first goal of the park and has a higher priory than other objectives such as Nature Conservation and Historic Resources Protection. That is not to say these objectives cannot be met while river control is a goal, but that stopbanks must fit in collaborate with river control (TDC, 2010).

3.4.2. Main Spring Grove Setting

The Main Spring Grove (MSG) stopbank was one of the first stopbanks constructed by the Nelson Catchment Board (now the TDC) in the late 1950s. The stopbank is 2.5 km long on the right side of the river, running mostly parallel to State Highway 6 between Wakefield and Brightwater.

The stopbank was built using a Caterpillar D4 Bulldozer to push gravel and dirt up from the river. In the 1970s willows were privately planted along the riverbank as a response to the gravel extraction by a landowner. Originally railway irons were used to tie the willows to the riverbank, since planting, the willows have been removed by new landowners although the railway irons remain.

The MSG stopbank now runs through twelve privately owned properties and a small section of Crown land. As a result it is not part of Council assets and no record of documents or maintenance is kept by the Council. The stopbank is set between 10 and 120 m back from the river bank.

The MSG stopbank aims to protect private properties from flooding. The private property protected is mostly used for livestock grazing, one property is growing hops.

3.4.3. Pitfure Setting

The Pitfure stopbank, unlike the Waimea and MSG stopbanks, was not constructed by the Council or its predecessors. It is believed from correspondence with the landowners that the stopbank was privately constructed in response to the 1982 and 1983 flood events. In 1982 and 1983 the Pitfure stream flooded significantly, inundating the nearby production land. One landowner responded by approaching the Council for consent to construct a stopbank who deemed it unnecessary. With the help of a contractor, the landowner built the entire length of the stopbank. The stopbank runs 1.5 km along the Pitfure Stream outside of Wakefield downstream to Bridge Valley Road. The stopbank was built using material from the Pitfure Stream bed that mainly consists of large cobbles and earth with fines. This construction method may have straightened, widened, and deepened the riverbed along the western section of the Pitfure stopbank, see Figure 3.4. Channel straightening can lead to faster stream flows that can cause further stream bank scour, thus further straightening the channel (CIRIA et al., 2013).

Figure 3.4.Typical West Pitfure stream channel bed section

The stopbank itself is divided in two by the Higgins Road Bridge that crosses the stream. The western half of the stopbank crosses three different sections while the eastern half runs through a single property. As with the MSG stopbank, production land is adjacent to the stopbank.

3.4.4. Confluence Setting

At the confluence of the Wai-iti and Wairoa Rivers is the Confluence stopbank. The Confluence stopbank is not in fact, continuous. Instead, a section of the stopbank runs along the Wai-iti River and a separate section is adjacent to the Wairoa River. The Confluence stopbank is contained entirely within the Council's Waimea River Park. It is believed the Confluence stopbanks were constructed during the same period as the Council stopbanks during 1963 and 1964.

3.5. Historical Flooding

3.5.1. Historic 1983 Observed Flooding

Historical flood records on both flow rates and flooding extents are hugely important as they provide documentation of actual flood events. This data allows us to see how flood frequency and extent have increase and decreased through time. For this study in particular, the flows and flood extent were used to recreate flood events.

The July 1983 event is the largest event on record for the Wai-iti Combined and Pitfure gauges. The storm lasted four days from the 8th to the 12th. The event was caused by:

> *"A moist northerly airstream brought prolonged heavy rain to northern areas of the South Island during the 8th and 9th of July 1983. The heavy rains resulted in extensive and severe flooding in parts of Marlborough, Golden Bay and Nelson Districts. Some 500 people were evacuated from residential areas at Tuamarina, Spring Creek and Renwick"*
>
> *Damage to dams, bridges, bridge approaches, river banks and stopbanks was extensive on all rivers in the Golden Bay, Waimea and Marlborough Counties" (Quayle, Pointer, & Challands, 1983)*

The heaviest rains from the event occurred in the high country of Tasman-Nelson, and Marlborough. During the storm, high country snow was rapidly melted and lead to the saturation of the ground soil which allowed for maximum runoff to occur. The heavy rain coupled with the snow melt lead to an increase in water levels. A landowner who was present along the MSG stopbank during the 1983 flood wrote in an email:

> *"Something happened that caused the bank to fail at Barton Lane in a significant flood. This caused the Wai-iti to flow across to Telenius Road & join with the Pitfure. The water across Telenius Road was about a metre deep. At 1:00 am on a Saturday night. I was coming home in my father's Chev. The car before me, a Ford Fairland headed into it & the headlights went under water, so I backed off & stayed in Brightwater. So that's what happens when the bank fails." (Higgins, 2018).*

These observations match with the flood maps the Council produced from the event.

3.5.2. Response to Historic Flooding

During the 1983 flood event, the Pitfure flooded as well as the Wai-iti and Wairoa. As a result of the Pitfure Stream flooding in the 1983 flood, a landowner, who had already suffered losses on their property the previous year, suffered more loss. This consequently caused the landowner to seek permission from the Council to develop a stopbank along the Pitfure Stream. It was understood that the Council at the time did not require permission or formal consent to be obtained. The construction of the Pitfure stopbank outside of Wakefield was therefore undertaken without formal consenting and the current stopbank remains unconsented.

The 1983 flood caused a significant amount of damage. During the event, about 26,000 stock were lost. From the event, insurance industry payouts were $2.3M ($7.5M in 2019 dollars) in Marlborough and Golden Bay (ICNZ, 2019). Within the study area, the Pigeon Valley Bridge was destroyed leading to the residents of Pigeon Valley becoming isolated. During the bridge collapse, the water supply pipes were also broken (NIWA, 2009).

4. METHODS

4.1. Condition Assessment Methodology

A condition assessment was completed as part of this study between the 28th and the 30th January 2019. The assessment took place along sections of the Council and undocumented stopbanks.

The key methods used for the condition assessment were:

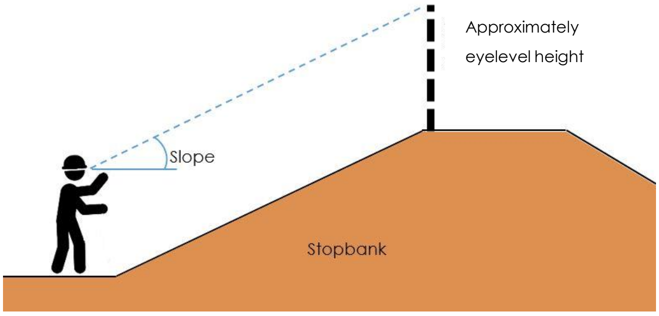
- Slope and height measurements of the stopbanks.
- A record of the stopbank cover and damage to notable structures built in/around the stopbank (such as power poles).
- Photographs documenting the above points.

The data collected by the condition assessment were documented using the condition assessment template in the appendix of the Bay of Plenty stopbank guidelines (BoPRC, 2014). This template covers works associated with drains, floodways and waterways, transitions and penetrations. The template can be used to identify and record problematic areas so they can be addressed.

The slope and height measurements were taken in an approximate manner and should not be any way be taken as precise measurements. Using an inclinometer smartphone software application at eyelevel, the slope to the top of an object approximately eyelevel height was measured, shown in Figure 4.1. This gave an estimation of the average stopbank slope.

The height of the stopbank was measured by using the inclinometer to look ahead at eye level. By moving to the point on the stopbank at eyelevel the next measurement could be taken. Repeating this until eyelevel was above the crest and then measuring the height to the crest, the height of the stopbank was determined. The methodology to do this is shown in Figure 4.1.

(a)



(b)

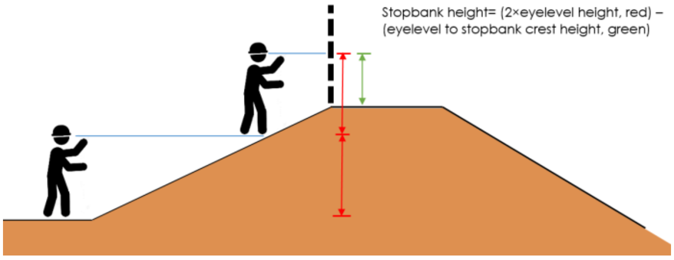


Figure 4.1. Condition assessment method for: (a) measuring stopbank slope (b) measuring stopbank height

Cross-sections were generally taken in areas that represented the new profile, not at regular intervals. The cover of the stopbank as well as structures present was noted at these locations.

Photographs were taken at cross-sections and at areas of significant surface damage, transition in stopbank cover (i.e. from woody vegetation to grasses), or at structures etc., these can be seen in Appendix B.

4.2. Flood Modelling

Flood mapping traditionally has been achieved using historical flood maps. However, as new technologies have become available, new methods to map flood have become available such as computational models. These models are able to estimate historical flood extents as well as predict flooding for user defined scenarios. Models simplify an area by reducing it to a grid of cells and then calculating the depth and velocity at each of the cells. This is calculated using governing equations at each time step until the simulation reaches its completion. The key data required to create a flood model are topographic data of expected inundation areas and flow inputs to control the volume of water entering the model. By modifying the input parameters, the flooding extent of different scenarios can be predicted. An example of this is increasing the amount of flow to predict the flooding extent under the effects of climate change.

4.2.1. Governing Hydraulic Equations

Flood models are governed by equations that calculate the velocity and depth that can operate in multiple dimensions. An equation set used to approximate 3-D flow is the Navier-Stokes equations. This equation set is not commonly used for flood modelling due to long model run times. Flood modelling in 3-D can be beneficial as it is capable of modelling hydraulically complex inlets (Swift, 2014).

The Navier-Stokes equations can be simplified to form the 2-D Saint-Venant equations. These are commonly used to describe 2-D flow because they can be applied to a wide range of uses such as mixed flow regimes, abrupt contractions and expansions, tidally influenced conditions, general wave propagation modelling and super elevation around bends (Farooq, Shafique, & Khattak, 2019). The key simplification between the 3-D and 2-D equation sets is that vertical variations in flow and velocity are neglected. This reduces the model run times, however it means that the model is unable to capture the depth varied velocities around structures. The assumptions of the Saint -Venant equations include (Betsholtz & Nordlöf, 2017):

- The pressure distribution is hydrostatic: vertical accelerations are neglected.
- Vertical variations in flow and velocity are neglected.
- The wavelengths are much larger than the water depth.
- The average channel bed slope is small.

The 2-D Saint–Venant equations are based around the conservation of mass and momentum. The Saint-Venant equations consist of one equation for conservation of mass and two for conservation of momentum, shown respectively as (USACE, 2016b):

$$\frac{\partial H}{\partial t} + \frac{\partial (hu)}{\partial x} + \frac{\partial (hv)}{\partial y} + q = 0, \tag{4.1}$$

$$\frac{\partial u}{\partial t} + u\frac{\partial u}{\partial x} + v\frac{\partial u}{\partial y} = -g\frac{\partial H}{\partial x} + v_t\left(\frac{\partial^2 u}{\partial x^2} + \frac{\partial^2 u}{\partial y^2}\right) - c_f u + fv, \tag{4.2}$$

$$\frac{\partial v}{\partial t} + u\frac{\partial v}{\partial x} + v\frac{\partial v}{\partial y} = -g\frac{\partial H}{\partial y} + v_t\left(\frac{\partial^2 u}{\partial x^2} + \frac{\partial^2 u}{\partial y^2}\right) - c_f v + fu \tag{4.3}$$

where H (m) is the water surface elevation, t (s) is the time step, h (m) is the water depth, u (ms^{-1}) and v (ms^{-1}) are the depth averaged velocity components in x and y directions respectively, q (m^{3}s^{-1}) is the source/sink flux term, g (ms^{-2}) is gravitational acceleration, c_f is the bottom friction coefficient, R is the hydraulic radius, and f is the Coriolis parameter.

The Saint-Venant equations can be simplified to form the Diffusion Wave equation where the dominating forces are gravity and friction. In this simplification, the unsteady, advection, and viscous terms in the momentum equation are disregarded. In general, the Diffusion Wave equation is faster to solve, and can reduce model instability. Because of this, it can also be used with a larger time step and still obtain stable and reasonably accurate solutions in cases where gravity and bottom friction are the dominant forces. A consequence of this simplification is that more uncertainty can enter the model as flow separation, eddies and momentum transfer cannot be accounted for. The Diffusion Wave equation is (USACE, 2016b):

$$V = \frac{-(R(H))^{\frac{2}{3}}}{n} \frac{\nabla H}{|\nabla H|^{1/2}} \tag{4.4}$$

where V (ms^{-1}) is the velocity vector, R (m) is the hydraulic radius, H (m) is the water surface elevation, n (sm$^{-1/3}$) is Manning's n, and ∇H (mm^{-1}) is the surface elevation gradient.

The 2-D flow equations can be further simplified to derive the 1-D Saint-Venant equations. 1-D flood models assume that water remains inside the river cross-section and only considers flow in the downstream direction. Additionally, terrain is represented as a sequence of the river and floodplain cross-sections perpendicular to flow direction (Ullah, Farooq, Sarwar, Tareen, & Wahid, 2016). Because of these limitations, 1-D models cannot simulate urban floods (Rahman, 2006). The benefit of 1-D models is that the simulation has a fast model run time. The 1-D Saint-Venant equations are based on the same principle as the 2-D equations of conservations of mass and momentum (USACE2016b):

$$\frac{\partial(Q)}{\partial x} + \frac{\partial A}{\partial t} + q = 0, \tag{4.5}$$

$$\frac{\partial Q}{\partial t} + \frac{\partial QV}{\partial x} + gA\left(\frac{\partial z}{\partial x} + S_f\right) = 0 \tag{4.6}$$

where Q (m^{3}s^{-1}) is the flow rate, A (m^2) is the cross sectional area, q (m^{2}s^{-1}) is the lateral inflow, t (s) is time and S_f is the friction slope, $\frac{\partial z}{\partial x}$ (mm^{-1}) is the water surface slope, V (ms^{-2}) is the average velocity and, g (ms^{-2}) is gravitational acceleration.

Alternatively, 1-D flows can be modelled using the 1-D kinematic wave equation. Since their inception in 1955 (Lighthill & Whitham, 1955), the kinematic wave equation has found widespread application in water sciences because it is simple, versatile, and reasonably accurate if the underlying assumptions are approximately satisfied. The theory couples the conservation of mass with a flux-concentration relationship. Because of this, the theory cannot accommodate waves that travel in the upstream direction as in the case of backwater flow (Singh, 2001). The kinematic wave equation is given as (Miller, 1984):

$$\frac{1}{c} \times \frac{\partial Q}{\partial t} + \frac{\partial Q}{\partial x} = 0 \tag{4.7}$$

where c is the kinematic wave celerity.

In the Saint-Venant equations, friction can be accounted for within the bottom friction coefficient. In this method, the term uses the Manning's n value, similarly to the Diffusion Wave equation. The Manning's n value is a measure of friction resistance water experiences while flowing over a surface. The inclusion of Manning's n in the equation sets allows energy losses to friction to be considered.

4.2.2. Software Selection

The Waimea catchment flows into a wide floodplain where much of the river reaches are relatively shallow. This floodplain lends itself to 2-D flood modelling as it has the potential to be extensively flooded and LiDAR data is available. LiDAR stands for Light Detection and Ranging and is a method of remote sensing that can be used to measure elevations, a more complete description of LiDAR can be found in Section 6.3.1. 2-D modelling also gives a greater understanding of flood velocities over 1-D as velocities are not assumed to be unidirectional. The ability to represent velocity with greater accuracy was beneficial to this study as velocity has been shown to influence monetary loss to residential buildings (Kreibich et al., 2009).

Many computer programmes are able to model in 2-D by implementing the equations seen in the previous section. As addressed in the literature review, many of these software packages are able to produce similar results as they all use the Saint-Venant equations or simplified versions of them (Environment Agency, 2013). The differences arise from:

- The mathematic formulation of the physical flood processes.
- The numerical method used to solve the mathematical formulation.
- The configuration of the numerical grid.

The software used to undertake the modelling in this study was the Hydrologic Engineering Centre's River Analysis System (HEC-RAS), developed by the US Department of Defence, Army Corps of Engineers (USACE) to manage the rivers, harbours, and other public works under their jurisdiction. HEC-RAS has found wide acceptance by many groups since its public release in 1995. Originally HEC-RAS was limited to 1-D modelling. The release of version 5.0 in February 2016 introduced 2-D modelling as well as sediment transfer modelling capabilities. The software has found commercial application in floodplain management and flood insurance studies to evaluate floodway encroachments. Some additional usages include bridge and culvert design, levee studies and channel modification studies. HEC-RAS version 5.0.5 was used in this study.

HEC-RAS is an implicit model. This means that the solution was found iteratively for each time step. HEC-RAS uses an Implicit Finite Volume Solution Algorithm and allowed for a larger computational time step (and therefore a faster simulation run time) than explicit methods. This method also provided more stability and robustness over traditional techniques.

The time step and the cell size of a model are related to each other by the Courant number. The Courant number is calculated for the Saint-Venant Equations using:

$$C = \frac{V\Delta T}{\Delta X} \qquad 4.8$$

where C is the Courant number, V (ms^{-1}) is the flood wave velocity, ΔT (s) is the computational time step and ΔX (m) is the average cell size.

For 2-D HEC-RAS models, the velocity is taken from each face, and the cell size is the distance between the two cell centres across that face. The Courant number is a measure of how much information travels across a cell in a single time step. The HEC-RAS 2-D modelling guide recommends that the Courant number is no larger than 1.0 with a maximum of 3.0 for the Saint-Venant Equations (USACE, 2016a).

The Courant equation shows an important relationship between cell size and computational time interval, and as a result, simulation runtime. As a cell size becomes smaller, the time step has to become smaller to satisfy the conditions of the Courant equation, and this leads to longer computation times. Selecting a cell size and time step is a balance between strong numerical accuracy while also maintaining a reasonable computation time.

4.3. Ungauged Hydrographs

There were several ungauged tributaries along the Wai-iti. These tributaries contributed significant amounts of flow during floods event. Because of this, it was important to recreate their flows within the model. Several methods were compared to find the most appropriate method of approximating the peak flood flow.

4.3.1. Scaled Method

This method approximated the size of the peak flow by scaling the peak flow of a gauged catchment based on area. This method was simple, although there is no consideration of catchment characteristics other than the area. The scaled method calculates the flow using (BoPRC, 2012):

$$Q_1 = Q_2 \times (\frac{A_1}{A_2})^{0.8} \qquad 4.9$$

where Q_1 (m^3s^{-1}) is the flow in the ungauged catchment, Q_2 (m^3s^{-1}) is flow in the gauged catchment, A_1 (m^2) is the area of the ungauged catchment and A_2 (m^2) is the area of the ungauged catchment.

The power is derived from a plot of mean annual flood and catchment area (Pearson & McKerchar, 1989), as seen in Figure 4.2. The plot shows MAF flow and area are related by a power of 0.8.

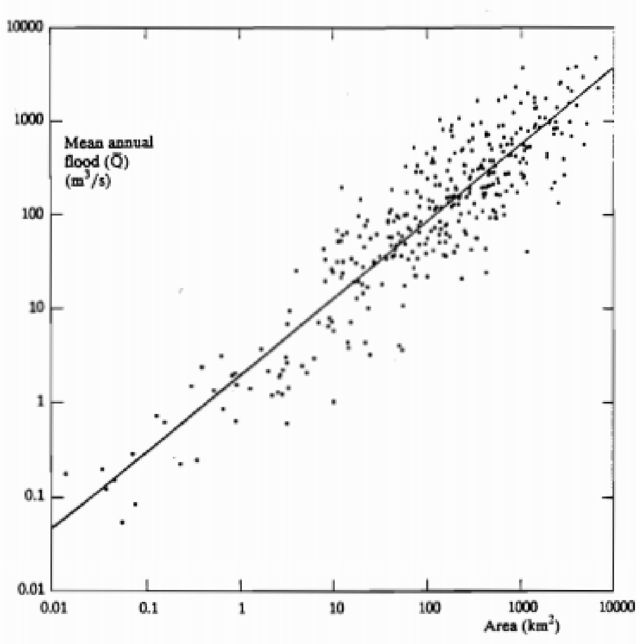


Figure 4.2. Mean annual flood verse catchment area. The fitted line has an equation of $Q = 2.04{\times}A^{0.808}$ (Pearson & McKerchar, 1989)

4.3.2. Regional Method

The regional method developed in 1989 provides contour maps relating MAF and catchment area (Pearson & McKerchar, 1989). The contour maps with the centroids of the ungauged catchments overlaid are shown in Figure 4.3.

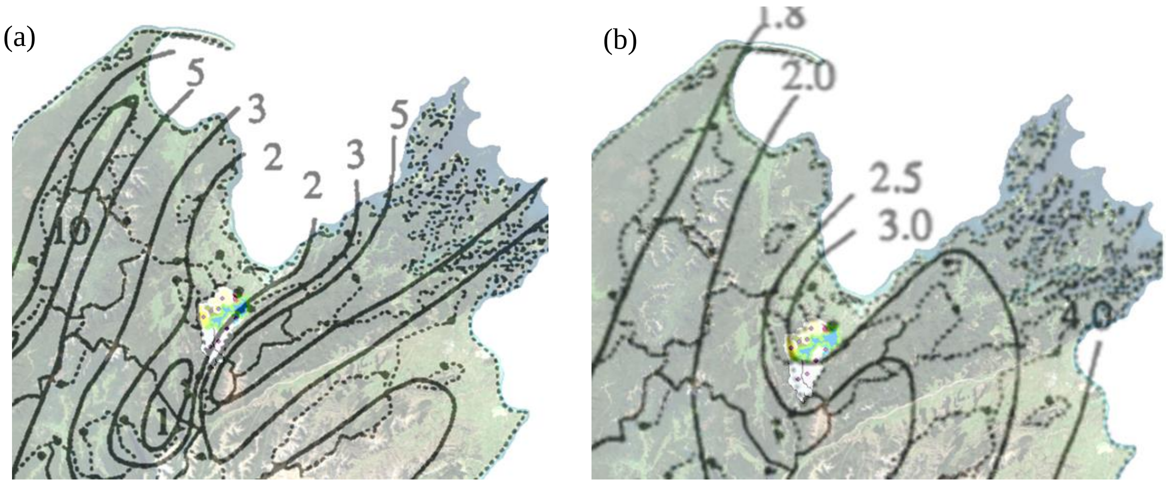


Figure 4.3. Regional method contour maps of: (a) specific discharge $Q/A^{0.8}$ (Pearson & McKerchar, 1989) (b) flood frequency factor q_{100} (Pearson & McKerchar, 1989)

This method accounts for the location of the catchment and is specific to New Zealand. However, estimation of the values for q_{100} and $Q/A^{0.8}$ are not exact as the catchments are not located on the contours. The equation for the regional method is:

$$Q_{peak} = Q_{mean\ annual}[X_T + (1 - X_T)q_{100}] \qquad 4.10$$

where Q_{peak} (m^3s^{-1}) is flow peak in the ungauged catchment, $Q_{mean\ annual} = (Q/A)^{0.8} \times A^{0.8}$, A (m^2) is the area of the ungauged catchment, $X_T = 1.1435 - 0.2486 \times Y_T$, $Y_T = -\ln(-\ln(1-1/T))$, T is the annual return interval and q_{100} is the flood frequency factor.

The regional method tended to underestimate flows in smaller catchments. Because of this, the method is not applicable to catchments less than 10 km^2. This may be because the contour plots are derived from studies of larger catchments (BoPRC, 2012).

4.3.3. Modified Rational Method

The modified method builds upon the regional method by accounting for surface cover by including a modification factor based on the slope. The modified rational can be applied to catchments larger than 50 ha but does not apply to catchments with any notable storage or backwater effects (BoPRC, 2012). The modified rational method equation is:

$$Q_{peak} = 1/360CIA \tag{4.11}$$

where C is the runoff coefficient, I (mm/hr) is average storm rainfall intensity and A (ha) is the area in hectares.

4.3.4. TM61

The TM61 method for estimating flood flows is the most comprehensive of all the method considered in this study. First published in 1953, and made a Public Works Technical Memorandum in 1955, it is now in its third revision. The TM61 method has been used in New Zealand throughout this time, and is still active in some parts of the country including the Bay Of Plenty Council (BoPRC, 2012).

The rational method estimates the peak flow rate of a catchment and can be applied to all catchment sizes. The TM61 method was originally developed for the explicit purpose of empirically estimating the design flood peak discharge in ungauged catchments (Ministry of Works, 1984). This method considers area, rainfall intensity, the shape of the catchment, and the cover and slope of the catchment. The equation used by the TM 61 method for calculating peak flood flow levels is:

$$Q_{peak} = 0.0139CRSA^{0.75} \tag{4.12}$$

where C is a coefficient that depends on the physiography of the catchment, R is the rainfall factor, S is the shape factor and A (km^2) is the area.

4.4. Model Accuracy Measurements

4.4.1. F-Value

An F-value was used to quantify the accuracy of a modelled flood extent against a historic flood extent. The F-value was used as the primary means of calibrating the model. The F-value compares the area that was flooded in both a model and a historical map with the areas of difference, i.e. areas where the model was flooded but not historically and vice versa. The formula for the F value is (Bates & De Roo, 2000):

$$F = \frac{\Sigma_i P_i^{D_1 M_1}}{\Sigma_i P_i^{D_1 M_1} + \Sigma_i P_i^{D_1 M_0} + \Sigma_i P_i^{D_0 M_1}} = 1 - \frac{\Sigma_i P_i^{D_0 M_1} + \Sigma_i P_i^{D_1 M_0}}{\Sigma_i P_i^{D_1 M_1} + \Sigma_i P_i^{D_1 M_0} + \Sigma_i P_i^{D_0 M_1}} \quad 4.13$$

where D is historical data, M is model data, P is pixel area, I is the pixel number, 1 indicates wets and 0 indicates dry.

An F-value of 1 indicates a perfect match between the historical and modelled flood extents. A cell was considered flooded if the average depth of the cell was more than 10 mm.

4.4.2. Nash-Sutcliffe

The Nash-Sutcliffe value was adopted to quantify the success of the model flow rates and stage heights. The Nash-Sutcliffe was found to be the best objective function for reflecting an overall fit of a hydrograph (Legates & McCabe, 1999). The Nash-Sutcliffe is widely used to evaluate the performance of hydrologic models. The Nash-Sutcliffe is defined using as (Nash & Sutcliffe, 1970):

$$NS = 1.0 - \frac{\Sigma_{t=1}^{T}(Q_o^t - Q_m^t)^2}{\Sigma_{t=1}^{T}(Q_o^t - \bar{Q}_o)^2} \quad 4.14$$

where NS is the Nash-Sutcliffe value, t is the time step, Q_0^t is the observed data at a time step, Q_m^t is the modelled data at a time step and $\bar{Q}_o$ is the mean of the observed data.

An Nash-Sutcliffe value of 1 indicates a perfect match between the historical and modelled flows. A Nash-Sutcliffe of zero indicates that the predictor of the model is as good as using the mean observed flow. This provides a convenient reference point for comparisons. A benefit of using the Nash-Sutcliffe was that it is sensitive to differences in the observed and modelled means and variances. However, it has been criticised for being overly sensitive to extreme values (Legates & McCabe, 1999). The Nash-Sutcliffe value was calculated at and from the historic Wai-iti Combined gauge and compared against the modelled data.

4.5. Development of the Maintained Scenarios

In order to simulate the maintenance of the undocumented stopbanks, the following method was developed.

In Arcmap, the DEM at the stopbanks were reduced to become level with the terrain at the landward side. The stopbanks were then rebuilt to a uniform height. This process created stopbanks of uniform heights that followed the natural rises in terrain.

The surveyed height of the Pitfure stopbank during the condition assessment was measured as approximately one metre high. This height was used as the maintained height of the Pitfure stopbank.

The Pitfure was also extended an additional 175 m until the stopbank joined a natural terrace that protected land upstream.

As the height of the MSG stopbank varied along its length due to undulations although generally the downstream and upstream end the stopbank were approximately 1.5 m and 2.6 m respectively. A uniform height of 2.6 m was used as the height for the maintained scenario. This was also the height of the Council stopbanks further downstream. The stopbank was also extended to prevent water flowing around the upstream end of the stopbank.

The result of this process was the creation of DEMs with the current MSG and Pitfure stopbanks replaced by maintained versions of themselves. The maintained stopbanks were a uniform height compared to the landward side. This removed undulations from the stopbank surface which were allowing water through the stopbank in other scenarios.

4.6. Riskscape Impact Analysis

Riskscape was used at part of this study to quantify the impact of the floods simulated in HEC-RAS. The Riskscape software was developed in collaboration by National Institute of Water and Atmosphere (NIWA) and GNS Science to assist organisations and researchers with estimating asset impacts and losses from natural hazards (Schmidt et al., 2011). The core components of a Riskscape analysis incorporate four different layers. These are assets, hazard, aggregation, and vulnerability.

The asset layer defined the assets which were damaged in the flood event. The asset layer used was the Tasman building assets layer that was specifically developed by the NIWA and GNS Research for Riskscape impact and loss modelling. Buildings, in this case, are defined as permanent enclosed structures.

The hazard layer was used to define the event, in this case, the hazard layer was the flood event simulated. The hazard layer was used to calculate hazard magnitude at the locations of the assets. The core aspects of a flood that cause damage are depth, velocity and duration. From HEC-RAS, the maximum depth, duration and velocity of each scenario were determined. Note that the maximums were over the entire simulation and not a representation of a single time step as different areas had maximum depths at different times. The maximum layers were inputted into Riskscape to define the depth, duration and velocity of the hazard layer of each scenario.

The aggregation layer was used to define the spatial boundaries from which results were grouped into. For this study, this layer was inconsequential as the domain was small enough to negate grouping and individual results could be analysed.

The vulnerability layer was used to select the measures of loss. Measures included in Riskscape cover a range of losses from building reinstallment costs to human losses. The measures used in this study included; damaged state, exposed start, functional downtime, human displacement and reinstallment cost. Riskscape software operates as a 'blackbox' because of this the exact definitions of the measures are unknown. Damage states, for example, are defined in Table 4.1. These definitions do not inform users of quantified definitions for insignificant damage or how irreparable structural damage is defined.

Table 4.1. Riskscape damage state definitions

Damage State	Riskscape Definition
Light	Insignificant damage, non-structural damage only
Minor	Light damage with possible minor non-structural damage
Moderate	Repairable structural damage
Severe	Irreparable structural damage
Critical	Structural integrity fails

Measures such as human susceptibility and human losses were omitted for the following reasons. Human losses were not reported in the results as in no event was a single person killed. In the worst-case event, approximately two people were in the 'no or light injury' category. This measure also required assumptions to be made about the warning time and the understanding of flood risk by residents which were beyond the scope of this project. The human susceptibility was left out as it required additional assumptions to be made surrounding the location and age of people that were outside of the scope. Within Riskscapek, human susceptibility also did not factor in the warning time which is believed to be significant in assessing the susceptibility to injury (Xichun et al., 2017).

It should be noted that the backend of the Riskscape software was inaccessible. As a result, it was not possible to access the fragility curves behind the software. This meant that it was difficult to evaluate the uncertainty of the numbers projected by the software.

4.7. Potential Impact Classification

The New Zealand Society on Large Dams has published a report that contains a method for assessing the consequences of a dam breach (NZSOLD & IPENZ, 2015). The key contributors to determining the PIC are the damage level and population at risk (PAR).

This method specifically excludes stopbanks and a complete potential impact classification (PIC) assessment was not carried out because it required detailed knowledge of dam breach mechanisms that were beyond the scope of this thesis. With this considered, the method was applied to determine a PIC for the stopbanks in this study because it gave an approximate indication of the stopbanks impact.

The method requires an assessment of the damage to; residential houses, critical infrastructure, natural environment, and community recovery time. Using this knowledge in conjunction with the damage definitions in Table 4.2 allows damage levels to be assigned to the various categories. The highest of these is used to determine the PIC.

The damage level to residential houses was determined using the outputs from Riskscape. In the NZSOLD method "destroyed" refers to uninhabitable for residential buildings. A building was considered uninhabitable if it was moderately damaged or worse in Riskscape. The definition of moderate damage in Riskscape was "repairable structural damage".

The damage to critical infrastructure was also considered, however in none of the design level events were any emergency, large industrial, commercial, or community facilities (the loss of which would have a significant impact on the community) flooded as a result of the stopbanks being removed. The level of damage to lifelines such as power, and telecommunication, which are part of critical infrastructure, was not considered. However, the impact on water supply and wastewater was assessed by examining the location of pipes and nodes using the Top of the South web application (NCC & TDC, 2019).

The community recovery time investigated the impact on downstream population centres, community facilities and inundated land covers. The assessment of the damage to the natural environment similarly considered land use as well as the impact to wetlands and other features of environmental significance (such as national parks, conservation areas and reserves).

Table 4.2. Method to determine damage level, adapted (NZSOLD & IPENZ, 2015)

DAMAGE LEVEL	SPECIFIED CATEGORIES				
	RESIDENTIAL HOUSES1	CRITICAL OR MAJOR INFRASTRUCTURE[2]		NATURAL ENVIRONMENT	COMMUNITY RECOVERY TIME
		DAMAGE	TIME TO RESTORE OPERATION[3]		
Catastrophic	More than 50 houses destroyed	Extensive and widespread destruction of and damage to several major infrastructure components	More than one year	Extensive and widespread damage	Many years
Major	4 to 49 houses destroyed and a number of houses damaged	Extensive destruction of and damage to more than one major infrastructure component	Up to 12 months	Heavy damage and costly restoration	Years
Moderate	1 to 3 houses destroyed and some damaged	Significant damage to at least one major infrastructure component	Up to 3 months	Significant but recoverable damage	Months
Minimal	Minor damage	Minor damage to major infrastructure components	Up to 1 week	Short-term damage	Days to weeks

1) *In relation to residential houses, destroyed means rendered uninhabitable.*

2) *Includes:*

   a) *Lifelines (power supply, water supply, gas supply, transportations systems, wastewater treatment, telecommunications (network mains and nodes rather than local connections)); and*

   b) *Emergency facilities - (hospitals, police, fire services); and*

   c) *Large industrial, commercial, or community facilities, the loss of which would have a significant impact on the community; and*

   d) *The dam, if the service the dam provides is critical to the community and that service cannot be provided by alternative means.*

3) *Estimated time required to repair the damage sufficiently to return the critical or major infrastructure to normal operation.*

Determining the PIC also requires the PAR to be assessed. This assessment is critical in determining the PIC that guides design and operation criteria. The definition of the PAR is the number of people likely to be affected by inundation greater than 0.5m in depth if a dam failure occurred. This includes people inside and outside of buildings. This is not always possible, and in these cases a reasonable estimate should be made based off factors such as; the size of permanent and temporary population and the times of occupancy.

It is also necessary to determine the potential loss of life. Fundamental factors in determining this include: Dam-break parameters, time or season of dam break, warning time, evacuation possibilities, and the willingness of people to evacuate.

Using the PAR and the highest damage level in conjunction with Table 4.3 allows the PIC to be found. Where multiple classification are possible, the potential loss of life is used to determine the PIC.

Table 4.3. Method to determine Potential Impact Classification, adapted (NZSOLD & IPENZ, 2015)

DAMAGE LEVEL	**POPULATION AT RISK (PAR)**			
	0	**1 to 10**	**11 to 100**	**More than 100**
Catastrophic	High potential impact	High	High	High
Major	Medium potential impact	Medium/High (see note 4)	High	High
Moderate	Low potential impact	Low/Medium/High (see notes 3, and 4)	Medium/High (see note 4)	Medium/High (see notes 2 and 4)
Minimal	Low potential impact	Low/Medium/High (see notes 1, 3, and 4)	Low/Medium/High (see notes 1, 3, and 4)	Low/Medium/High

Notes:

1. *With a PAR of 5 or more people, it is unlikely that the potential impact will be low.*
2. *With a PAR of more than 100 people, it is unlikely that the potential impact will be medium.*
3. *Use a medium classification if it is highly likely that a life will be lost.*
4. *Use a high classification if it is highly likely that 2 or more lives will be lost.*

5. STOPBANK CONDITION ASSESSMENT

A stopbank condition assessment was undertaken to investigate the condition of the stopbanks and ground-truth the assumptions of the HEC-RAS model. A deeper knowledge of the stopbank conditions gave a more complete understanding of the modelling results. The condition assessment involved creating a geospatial data set containing information and physical characteristics of the undocumented stopbanks.

A review of Council documentation on the privately-owned stopbanks was carried out. This stage identified the locations of undocumented stopbanks that the TDC was informally aware of. Council documentation of the undocumented stopbanks was understandably limited as the Council had no formal knowledge of the stopbanks' current condition. This raised the question: were the stopbanks being maintained to a standard where they would be able to route a flood and perform their function?

LiDAR revealed the locations of the undocumented stopbanks and Land Cover Database indicated the land use. However, these sources did not show the condition of the stopbanks. As a result of the information gap, field research was completed to catalogue the characteristics of the undocumented stopbanks.

The field condition assessment was completed between the 28th and the 30th January 2019 along the lengths of the Council and undocumented stopbanks. The data collected corresponded to photos of the stopbanks' condition, condition assessment forms, and approximate cross-sections. The method for obtaining the data is found in section 4.1. The cross-section details can be seen in Appendix A. The locations and details of the access roads, structure and surface damage can be found in Appendix B.

It was difficult to characterise the condition of the undocumented stopbanks as the amount of vegetation and amount of surface damage varied greatly along their length. This made it problematic to generalise the current condition of the stopbanks as a whole. The following sections discuss stopbank condition guidelines and assess the individual stopbanks' condition with regard to cover, structures present, surface damage and roads, cross-sections of the stopbanks are also provided.

5.1. Stopbank Condition Guidelines

5.1.1. Damage Sources and Impacts

The cover of a stopbank is important as the vegetation contributes to the structural integrity and therefore the overall performance of the stopbank (CIRIA et al., 2013). Best practices for cover maintenance aim to ensure that the stopbank is resistant to erosion from wind, rain, traffic, and in the event of flooding, overtopping, and scour. Without protection from these, the stopbank may become more vulnerable to levee failures. The most common surface protection is a robust grass covering that can provide stability and protection from erosion. Maintenance of the cover is also necessary. Regular mowing adds cut grass to the stopbank batters, aiding grass growth and providing stability via the roots. Grass cover additionally reduces the moisture loss which improves the stability of the stopbank.

Although grazing is an efficient method for grass trimming and maintaining root systems, grazing with cattle can cause surface rutting. Ruts are defined as a long stretch of depressions that can range in size from smaller than a centimetre to larger than 0.3 metres (CIRIA et al., 2013). If a rut is larger than 0.15 m according to the USACE levee inspection checklist, repairs need to be undertaken (USACE, 2008). Ruts are an issue because they disturb the surface covering and allow water to pond and seep into the stopbank. The increase in moisture content may decrease the strength of the stopbank and trigger a failure from slope instability in a worst-case scenario. By disrupting the stopbank surface, ruts also expose the stopbank to more surface erosion that can result in piping as explained earlier. The BOP guideline recommends against allowing livestock to graze along stopbanks for this reason (BoPRC, 2014).

Cattle may also cause depressions in crest height. Depressions are weak points that can cause overflow before the design level of a stopbank is reached. This type of damage can expose the soil to further deterioration. Because of the reduction in crest height, a stopbank may be unable to meet its designed performance.

Trees can be particularly detrimental to stopbanks. Generally speaking, tree roots spread further from the trunk than the canopy branch so the roots can find water and nutrients. From the surface, trees may not appear to cause damage, however, under the surface the root systems may potentially be weakening the structural integrity.

Fallen trees may also rot creating large voids with the stopbank. These may weaken the performance by exposing the stopbank to rainfall, and further erosion that can allow a breach failure to occur. Voids that lower the crest may also allow overtopping to occur, lowering the capacity of the design.

A bramble cover represents a hazard to the stopbanks performance as it establishes wide-spreading root systems. Brambles can deteriorate the surface of the stopbank by blocking sunlight, absorbing nutrients which discourages grass growth as well as encouraging external erosion (CIRIA et al., 2013).

Structures built into stopbanks can sometimes lead to their failure. Penetrations into the groundwater layer within the stopbank can cause internal erosion that may result in stopbank failure. Alternatively, penetrating the seepage path on the downstream stream slope can cause a piping failure (CIRIA et al., 2013).

Installing pipelines within stopbanks is undesirable because it is difficult to compact the earth to a sufficient level around the pipe and may lead to failure (FEMA, 2005). When the uncompacted soil is subjected to hydraulic pressures greater than the internal soil pressure, the soil fractures. Water flowing through the fracture can erode its sides. This creates a preferential path for seepage between the pipe surface and stopbank. This seepage can ultimately result in a piping failure and the collapse of the stopbank (BoPRC, 2014). The USACE standards recommend installing drainage fill on the downstream inlet, one third the length of the pipe (USACE, 1992).

Similarly, structures within the river that restrict flow such as fallen trees can be undesirable as they increase the water levels, and therefore the risk of overtopping. Such blockages can increase the amount of local scour that may destabilise stopbank slopes. Substantial scour of stopbank toes or bank caving can cause instability. Instability occurs when the active forces on soil particles exceed the resistive forces of the soil. Instability is related to failure mechanisms such as rotational/translational sliding, tilting and settlement. (CIRIA et al., 2013).

Incorporating a road into a stopbank design is sometimes necessary as it provides access to the upstream slope during flooding. A consequence of building a road into a stopbank is that the crest is required to be a minimum 3 m wide to safely accommodate a vehicle. The additional loading and wear from vehicles can cause additional deterioration of the stopbank crest, creating a need for more frequent crest maintenance. The maintenance cost can be reduced by applying gravel or asphalting the surface of the road (BoPRC, 2014).

5.1.2. Cross-sectional Guidelines

The BOP guidelines recommend a maximum slope of 2.5H: 1V for the upstream slope and 2H: 1V for the downstream slope. These maximums are suggested based on soil type. Another constraint is the construction of the stopbank. 2H: 1V is accepted as the steepest slope that can be easily constructed. However from a maintenance perspective, a ratio of 3H: 1V is recognised as the steepest slope that can be easier mown with conventional equipment (BoPRC, 2014). The USACE 2008 recommends a minimum slope ratio of 3H: 1V for new stopbanks (CIRIA et al., 2013).

The Bop guidelines have set tolerance levels that state the crest width should not increase or decrease more than 0.2 m from the design width (BoPRC, 2014).

5.2. Council Stopbanks

5.2.1. Cover

The Council stopbanks ran 10 km along either side of the Waimea River and the downstream ends of the Wai-iti and Wairoa Rivers. A sample of the Council stopbanks were assessed during this study, consisting of approximately 2 km of each side and at the upstream and downstream ends of the stopbanks. As part of the maintenance scheme the stopbanks were mowed two to three times annually. At the time of the assessment, the Council had delayed the mow due to the fire risk posed by the hot dry summer. This meant that the stopbanks had more vegetation than usual. However, the Council stopbanks generally were uniformly covered by large grasses and clear of significant vegetation. A typical cover is shown in Figure 5.1. It should be noted that there were trees planted at the toes of the stopbank at three locations; at one of these, roots were present on the stopbank crest surface.

Figure 5.1. Typical cover of a Council stopbank

5.2.2. Structures

Within the Tasman region, shallow water bores supply water to Mapua, Richmond, and Stoke. As part of this, six pump stations and one discharge point have been built into the Council stopbanks downstream of the Appleby Bridge, (NCC & TDC, 2019).

Other structures built into the stopbanks consisted of two powerlines and a conduit. It was unknown how deep the powerlines penetrated into the stopbanks. The flap on the conduit suggested that it was used to drain water from the landward side during a flood. The flap itself was clear of excess vegetation. Additional research into their soil mechanics and the construction methods is needed to determine if these structures along the Council stopbanks are degrading the structural integrity.

5.2.3. Surface Damage

There were no signs of subsidence or surface cracking along the entire length of the surveyed Council stopbanks

5.2.4. Roads

The Council stopbanks were wide enough to accommodate standard vehicle passage, with a minimum crest width of 3.0 m. The stopbank on the true right side of Waimea, although wide enough to be driven on, did not appear to have been used for this purpose regularly.

On the true left, a road ran along the crest. This road was not available for public use. The road in some sections was gravelled.

Along the stopbanks, there were three access crossings, one of which was damaged. The road did not have a gentle slope and appeared to be used by motorcycles resulting in rutting of the stopbank slope cover.

5.3. Main Spring Grove Stopbank

The Main Spring Grove (MSG) stopbank ran from the start of Spring Grove and terminated at the outskirts of Wakefield. The entire 2.5 km of the MSG stopbank was assessed for damage and other significant features. Before this survey, the Council had no official knowledge of the condition of the MSG or other undocumented stopbanks' condition, as they were under private ownership.

5.3.1. Cover

The MSG stopbank was approximately 2.5 km long, running from Brightwater to Wakefield. The cover of the MSG stopbank varied across its length from a grazed grass cover to trees growing between impenetrable blackberry bushes. Roughly 50%, and 40% of the stopbank cover were grasses, and blackberry/impenetrable vegetation respectively. The grassed sections were similar to those seen along the Council stopbanks. In two of the properties, cattle were seen grazing.

Impenetrable cover for this study was cover that was difficult to transverse comfortably. In most cases this was due to brambles or the density of large vegetation.

To remove the widespread bramble along the undocumented stopbanks would require either digging up the entire surface to remove the roots or first mowing the banks and then spraying herbicides to kill the new shoots. Removal of the roots would require a significant amount of funding and is likely to cause damage to the stopbank. This type of cover has associated issues similar to the trees, discussed in section 5.1.1.

Figure 5.2. Example of brambles along the Main Spring Grove stopbank

Because MSG stopbank was not maintained by the Council it was not mown as part of the Council maintenance scheme. The observed lack of mowing had allowed larger vegetation to take root along the stopbank. It was difficult to definitively express the damage this vegetation had on the stopbank due to the variations in growth and density from each location. Toward the Brightwater end of the MSG stopbank, there were large trees growing on the upstream toe. In another segment of the stopbank, a grove of trees had grown, stretching approximately 150 m long.

5.3.2. Structures

Along the MSG the only structures observed were several power poles built into the top of the stopbank and two pipes. As with those in the Council stopbank, it was uncertain how deep the powerlines penetrated into the stopbank.

The MSG stopbank also had two pipes through it at different locations. These were used to discharge excess surface water from adjacent production land to the Wai-iti River. The pipes were fitted with wooden flaps to prevent backflow. Drainage fill installed on the downstream inlet was not observed in this case.

5.3.3. Surface Damage

The MSG stopbank had over ten accounts of surface damage. Three areas of subsidence were located at access roads, and in two of these the damage was the result of vehicle use. In two other locations, cattle were able to graze the stopbank batters. The largest depression was 0.7 m deep.

5.3.4. Roads

There were three access roads across the MSG stopbank. All of these were located at the Brightwater end of the stopbank so that the production land could be accessed on the waterside of the stopbank.

5.4. Pitfure Stopbank

The Pitfure stopbank ran along the Pitfure Stream from outside of Wakefield for approximately 1.5 km. The entire length of the Pitfure was surveyed for damage.

5.4.1. Cover

The cover of the Pitfure stopbank west of the Higgins Road Bridge was dominated by grasses surrounded by trees and large brush; the typical cover of the West Pitfure is shown in Figure 5.3. Throughout the length of the West Pitfure, trees had fallen and their roots had rotted, leaving large voids. These represented the most significant form of damage to the Pitfure stopbank. The largest void measured 1.2 m deep. In stark contrast, east of the Higgins Road Bridge the typical cover of the Pitfure stopbank was grasses similar to those seen along the Council stopbanks, Figure 5.3.

(a)  (b) 

Figure 5.3. Example of the typical cover of the: (a) West Pitfure stopbank (b) East Pitfure stopbank

5.4.2. Structures

Along the length of the Pitfure, there were four pipes across the stream and a conduit pipe. The pipe bridge structures were implemented by landowners to create pipelines across the stream. These structures penetrated the stopbank surface

Along the east side of the Pitfure stopbank, a blockage structure from sheets of metal had been created. The purpose of this structure was unknown but blockages were undesirable. Localised scour was observed at this location. At this location, the outside edge of the stream was deeper and the outside bank had experienced scouring.

5.4.3. Surface Damage

Bank erosion was a large issue along the eastern side of Pitfure stopbank. This scour was possibly a result of the stopbank construction method. As explained in section 3.4.3, the stopbank was built using material from the Pitfure Stream. During the construction process, it is possible the stream was deepened, widened and straightened. These processes may have caused higher flows that could have progressively scoured the stopbank slopes until the stopbanks reached their current condition. This type of erosion was prevalent for approximately 500 m of the Pitfure stopbank. If the banks scour further, it is likely that the slopes of the stopbanks will be eroded entirely, removing the stopbank and any of its protective properties.

Along the eastern side of the Pitfure stopbank, there were numerous areas of undulations. These undulations were similar to those seen in areas of the MSG stopbank and carry similar issues of overtopping and erosion.

5.4.4. Roads

There were three access roads over the Pitfure stopbank. These roads were located along the stopbank on the east side of the Higgins Road Bridge. The Pitfure stopbank itself was bisected by Higgins Road Bridge. At this intersection, the road rose above the crest of the stopbank. This meant that the road did not compromise the stopbank by reducing the crest height.

5.5. Confluence Stopbank

The Confluence stopbank ran between the downstream ends of the Wai-iti and Wairoa Rivers. The stopbank was built on part of the Waimea River Park.

5.5.1. Cover

The first 200 m of the Wairoa side of the Confluence stopbank was covered in newly planted bush, shown in Figure 5.4. Because the land was part of the Waimea River Park Management Plan it was subjected to the park objectives (TDC, 2010). The primary objective of the park was river control and soil conservation.

> *"Manage the riverbed and berm lands within the park to protect surrounding lands from flood flows of the Waimea River, to assist in maintaining the Waimea Plain aquifer and to maintain water quality"*

The second objective of the park was nature conservation:

> *"Manage the riverbed and berm lands to protect existing areas of wildlife habitat and to restore, wherever practicable, indigenous vegetation and habitats within the park, so long as such management is compatible with river control and soil conservation"*

This newly planted wildlife habitat was planted by the TDC Parks and Reserves team as part of fulfilling the second objective of conserving nature and restoring indigenous vegetation with the park. It should be noted that this area runs alongside the Twin Rivers Walkway.

Figure 5.4. Example of newly planted trees along the Wairoa Confluence stopbank

Further downstream on the Wairoa side the stopbank has a cover of grasses, brush and, blackberry and has the associated risks of woody vegetation mentioned previously.

The Confluence stopbank was discontinuous, stopping after 400 m, parallel to the landowner's house on the Wairoa side. The stopbank begins again on the Wai-iti side of the confluence. On this side, the stopbank was covered by gorse and broom. This essentially gives the stopbank an impenetrable cover.

Removal of broom and gorse is difficult as both are able to regrow from roots and pulling the roots would require significantly disruption of the stopbank material.

5.5.2. Structures

Along the total 2.3 km of the Confluence stopbank, no structures built into the stopbank were observed.

5.5.3. Surface Damage

Although not specifically surface damage, the material used to construct the stopbank was uncompacted sand, silt gravel where there were newly planted trees. This type of material is easily eroded and unlikely to provide serious erosional resistance during a flood event.

Further downstream and along the Wai-iti confluence area, the impenetrable vegetation made it difficult to gauge the surface damage. There was no observed damage to the Confluence stopbanks, however this does not mean there was no damage present.

5.5.4. Roads

The Confluence stopbank was the only stopbank surveyed that was open to public access. Along the Wairoa segment, there was a cycleway that was part of the 'Tasman Great Taste Cycleway'. The cycleway was along a road that was mostly on the landward side of the stopbank. The road was gated and could not be accessed by public motor vehicles.

There were three access roads on the Wairoa side of the stopbank. At this location there was a significant drop in crest height, Figure 5.5. The adjacent stopbank was roughly 3 m high. The access road therefore essentially created a 3 m depression.

Figure 5.5. Example of access road interrupting the Wairoa Confluence stopbank

5.6. Typical Cross-Sections of Stopbanks

In developing a geospatial data set of the characteristics of the undocumented stopbanks, rudimentary cross-sections were taken along the lengths of the stopbanks. It is hoped the information from these cross-sections can be added to the New Zealand Inventory of Stopbanks (NZIS) to raise awareness of undocumented stopbanks (Blake et al., 2018).

The cross-sections measured the height of the upstream and downstream slopes as well an average slope angle using an improvised inclinometer, see section 4.1. The angles measured were taken to give an estimate and not to be used as definitive final numbers The cross-sections show the changing profiles of the stopbanks. Before this survey, the Council only had LiDAR data to give an indication of the stopbank profiles. The survey data will help inform the Council about the undocumented stopbanks' dimensions.

5.6.1. Council Stopbanks Cross-sections

The Council stopbanks were originally designed in the early 1960s to contain a 50-year flood while maintaining 0.6 m of freeboard. The Council stopbanks had an average height of 2.6 m and were the most uniform of the stopbanks surveyed.

The average slope angle of the Council stopbanks was approximately 2.1 H: 1V. Given the uncertainty of the measurements taken, is it possible that the actual ratio could lower than 2.5H: 1V. The ratio measured indicates that the slopes of the stopbanks were within the accepted range.

The access roads across the Council stopbanks had decreased slopes so that vehicles could safely navigate to the other side, reducing the risk of a bearing capacity failure due to the additional loading. The crest of the Council stopbank was on average 3.2 m wide. This was wide enough to support a vehicle as suggested in the international stopbank guideline and the BOP guidelines. The road along the road is described earlier in 5.2.4.

5.6.2. Main Spring Grove Cross-sections

The MSG stopbank had a cross-section that fluccuated in height. Toward the Brightwater end of the stopbank, the stopbank slopes were approximately 1.2 m high. The crest along this section had an undulating surface of ± 0.2 m

Toward Barton Lane, the upstream and downstream heights of the stopbank were 2.3 m and 1.6 m respectively. The upstream height was difficult to definitively measure as the upstream slope was covered by impenetrable brambles leading down to the river bed. It was therefore difficult to define an exact stopbank toe. South of Barton Lane the up and downstream height rises to 2.5 and 1.5 respectively.

The downstream slopes of the MSG stopbank varied between 2.7 H: 1V and 2H: 1V, which were within the recommended limits for the maximum stopbank slope angle but outside the recommended slope limit to be easily mowed. The upstream slope had a more variable slope angle ranging from 1.4H: 1V to 4.7H: 1V. The slope was also difficult to measure. As previously mentioned the toe of the stopbank was not always well defined due to cover conditions. In many areas, the toe often blended into the natural bank that slopes toward the riverbed. The best estimate of the toe location was used when measuring the average slope angle.

The crest width varied from 0.4 m to 2.0 m along the stopbank, this was outside of the recommended limits for crest width variation (BoPRC, 2014).

5.6.3. Pitfure Cross-sections

The Pitfure stopbank was the lowest of the stopbanks surveyed. The crest of the Pitfure stopbank west of Higgins Road was characterised by an undulating surface of ± 0.5 m. This was quite significant as the average downstream slope height was 0.9 m. These undulations were seen for the first 400 east of Higgins Road, past this point the stopbank lowered to a roughly 0.5 m bund that did not have a crest. The upstream height of the stopbank was measured from the channel bed that sloped up to the crest and had an average upstream slope height of 2.6 m.

The slope angles of the Pitfure stopbank were 2H: 1V. This meant the Pitfure stopbank was below the BOP guidelines for maximum slope angle on the downstream side, but not for the upstream slope.

East of Higgins Road, the stopbank crest elevation no longer undulated as it did in the west segment. The heights and slopes of the stopbank remained similar to those along the western half. However, after approximately 550 m after the panel blockage, the sides of the stopbank were scoured in many places to a sheer face. After this point, the stopbank repeatedly stopped and started.

5.6.4. Confluence Cross-sections

Along the Wairoa section of the Confluence stopbank there was an initial height of 0.8 m on either side. This extended along the segment with newly planted cover. After this segment, the stopbank ended and began on the other side of the cycleway. On this side, the stopbank was larger with a height of 3 m and sheer sides. The upstream side of the stopbank was terraced land.

The stopbank was then briefly interrupted by an access road and began again, continuing until it finished adjacent to a house located between the Confluence stopbanks. At the downstream end of the Wai-iti Confluence stopbank, the upstream and downstream height of the stopbank rose to 2.1 m and 1.1 m respectively. The stopbank continued in the manner until it terminated near landowners buildings.

There appeared to be a second stopbank at the upstream end of the Wai-iti side of the confluence that was closer to the river. This stopbank appears to be intentionally constructed and not formed by natural processes. This second stopbank bank was approximately 400 m long with a height of 1.8 m and 2.0 m upstream and downstream respectively. This stopbank was not attached to the longer Wai-iti Confluence stopbank.

5.7. Summary

The undocumented stopbanks were diverse in height and cover. Observations were grouped into smaller reaches. The properties of the major stopbank sections are generalised in Table 5.1.

Table 5.1. General condition along stopbank segments

STOPBANK	CREST WIDTH (m)	SLOPE HEIGHT (m)	SLOPE ANGLE (°)	COVER	SIGNIFICANT ISSUES
Council	3.2	2.6	26	Grasses	-
MSG	1.1	1.2	23	Grasses - Impenetrable	Woody Vegetation
Pitfure West	1	1	26	Woody Vegetation	Woody Vegetation, Decayed Roots
Pitfure East	1.2	1	33	Grasses	Scour of Banks
Confluence Wai-iti	3.4	1.5	23	New planting-Scrub	Access Roads
Confluence Wairoa	1.7	1.5	23	Impenetrable	Woody Vegetation

Of the stopbanks surveyed, the Council stopbanks were in the best condition. The Council stopbanks also had a clear maintenance scheme that included activities such as being regularly mowed.

In contrast, maintenance of the undocumented stopbanks was at the landowner’s discretion. The undocumented stopbanks were also found to have more areas of woody vegetation, more instances of surface damage, and a wider distribution of stopbank heights. In some areas, vegetation such as bramble and broom had grown, making the cover impenetrable. This may have been because the landowners did not understand the risk vegetation poses. One landowner remarked that he believed that vegetation helped to bind the soil together, making the stopbank stronger. Other landowners had allowed cattle to graze sections of their stopbanks. Grazing removed the woody vegetation, but depressions were observed in these areas.

During the construction of the Pitfure stopbank, the Pitfure Stream may have been straightened, widened, and deepened, which may have resulted in faster flows that may have caused the large amount of scouring observed on the East Pitfure stopbank slopes.

The damage along the Pitfure and Confluence stopbanks means that it is likely sections will not provide protection to the level to which they were constructed. As a result, the adjacent land may flood in a lower return period event than designed to withstand.

6. MODEL INPUTS

A flood model was created to evaluate the impact of the undocumented stopbanks. The inputs to the flood model consisted of flows to the main river channels, terrain, and channel bathymetry. The model was developed with a cell size of 12 m and a time step of three seconds. The initial distribution of water was determined by allowing the model a warm up period. Within the warm up time, the flows were slowly increased over 25 hours to the initial flow levels before the flood. This length of time allowed ample time for the water to reach the lower boundary. These components created the initial model which was calibrated, validated and used to assess the impacts of the undocumented stopbanks. The model did not include the effects of human actions such as sandbagging and pump stations.

6.1. Model Equations

HEC-RAS offered two equation sets to determine the flow rate through cells, the Diffusion Wave equation and the Saint-Venant equations. The governing equations for these are outlined in section 4.2.1.

It was chosen to work with the Diffusion Wave equation initially because it allowed for a shorter simulation runtime. However, preliminary modelling revealed that it was critical to use the Saint-Venant equations. Figure 6.1 shows a simulation with the same parameters except for the equation set. The figure illustrates that there was significantly more similarity between the historic and modelled flood extents, and therefore better model performance, when the Saint-Venant equations were used. 'Model Union' is the area inundated in the model and historical flood extent. 'Model Union' is not linked to depth as that data does not exist for the historical flooding. It is not understood why this system relied on the Saint-Venant equations to be modelled but it appeared consideration of momentum was essential.

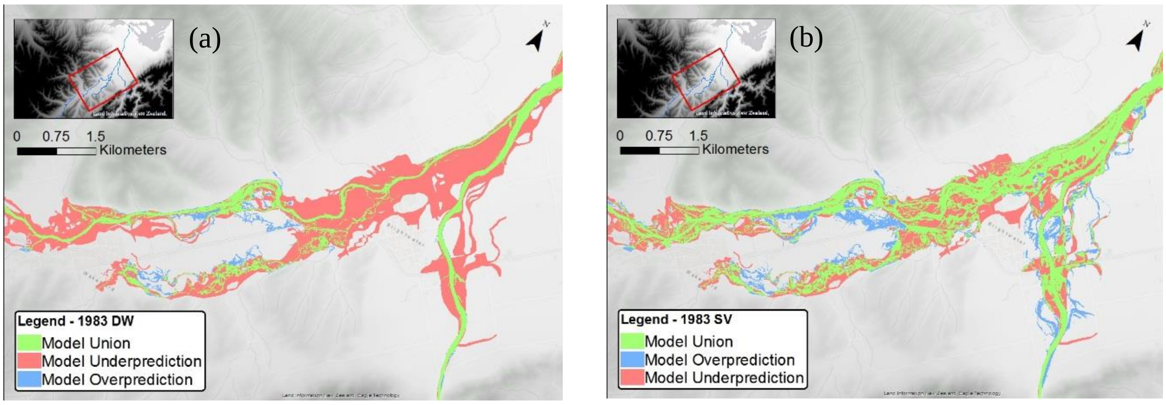


Figure 6.1. Preliminary model performance of 1983 flood event: (a) using the Diffusion Wave equation (b) using the Saint-Venant equations

The HEC-RAS user manual (USACE, 2016a) states that if there are significant differences when the different equation sets are used, the Saint-Venant equations should be assumed to be more accurate, and future simulations should use the Saint-Venant Equations.

6.2. Flow Boundary Conditions

Flow boundary conditions are a set of constraints that include inflows and outflows. Flow boundary conditions govern the total volume of water in the model and the rate at which it enters and exits the domain. Stage and flow recording devices are considered good locations for flow boundary conditions as they act as a point of observed data. However, recording devices are not always present and within these catchments the flow was ungauged. Rainfall was not considered in this study.

6.2.1. Gauged Catchments

TDC gauges were used for flow boundaries where possible. Within the study area, there were five gauges of relevance. Four of these sites were used as boundary conditions at the edge of the model domain. The other gauge was used as a validation point for comparing stage and flow data. The gauge locations are shown in Figure 6.2.

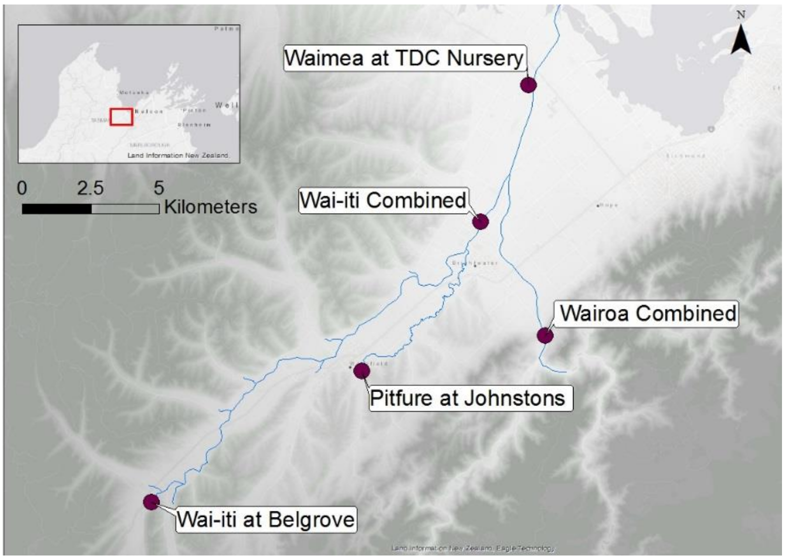


Figure 6.2. Gauge locations within the Waimea floodplain

In November 1957 a gauge was installed at the Wairoa Gorge, this made it the second longest-running gauge the Council maintains. In 1992 the gauge was relocated slightly further upstream. The records have been well-maintained and provide 60 years of reliable data for flow analysis. The mean annual flow of the Wairoa River was 16 m^3s^{-1}. The combined record of these two gauges shall henceforth be referred to as the Wairoa Combined Gauge. This gauge was the upstream flow boundary condition for the Wairoa River.

The Wai-iti River has gauges at two locations. The Wai-iti at Belgrove gauge was installed in December 1983 for flood warning purposes. At this gauge, the bed was unstable. Previously weirs have been constructed to help regulate the flow of the river, however, most of these weirs have failed by “blowing out” (TDC, 1983). This gauge was the upstream flow boundary condition for the model.

Further downstream, in 1976, a gauge was installed at the Brightwater Bridge. Ten years later the gauge was relocated 850 m downstream to its current location at Livingston Road. The average annual flow for the Wai-iti River was 4 m^3s^{-1}. The combined record of these two gauges shall be referred to as the Wai-iti Combined Gauge. This gauge was used for the validation of model flow and stage readings.

The Tasman District Council Tree Nursery gauge near Appleby Bridge was first installed in February 1998 to measure the losses to the groundwater downstream of the Wairoa River. Due to technical reasons, the gauge was removed and reinstalled in 2001, and again in 2004. The stage from this gauge acted as the downstream boundary condition for the model.

The Pitfure gauge was installed in August 1982 outside of Wakefield, however, after five years, the site was closed and the gauge removed. This gauge acted as an upstream boundary condition for the Pitfure Stream where possible.

The periods the different gauges were active is summarised in Table 6.1.

Table 6.1. Waimea floodplain gauge details

River	**Gauge Location**	**Record Length**	**Catchment Upstream (km^2)**	**Q_{90} (m^3s^{-1})**	**Q_{10} (m^3s^{-1})**
Wairoa	Wairoa at Gorge	1957-1995			
	Wairoa at Irvine's	1992-2018			
	Wairoa Combined	1957-2019	462	2.74	31.4
Wai-iti	Wai-iti at Brightwater Bridge	1976-1986			
	Wai-iti at Livingston Road	1986-2019			
	Wai-iti Combined	1976-2019	285	2.2	8.5
	Wai-iti at Belgrove	1986-2019	61	0.2	2.5
Waimea	**Waimea at TDC Nursery**	1998-2019	772	1.9	39.2
Pitfure	**Pitfure at Johnston's**	1982-1987	11.5	0.0	0.4

The flows over time at the gauges are described in Figure 6.3. The figure illustrates that further downstream the flood peaks become larger in magnitude. This is because further downstream more tributaries contribute to the flow, leading to the higher peaks seen.

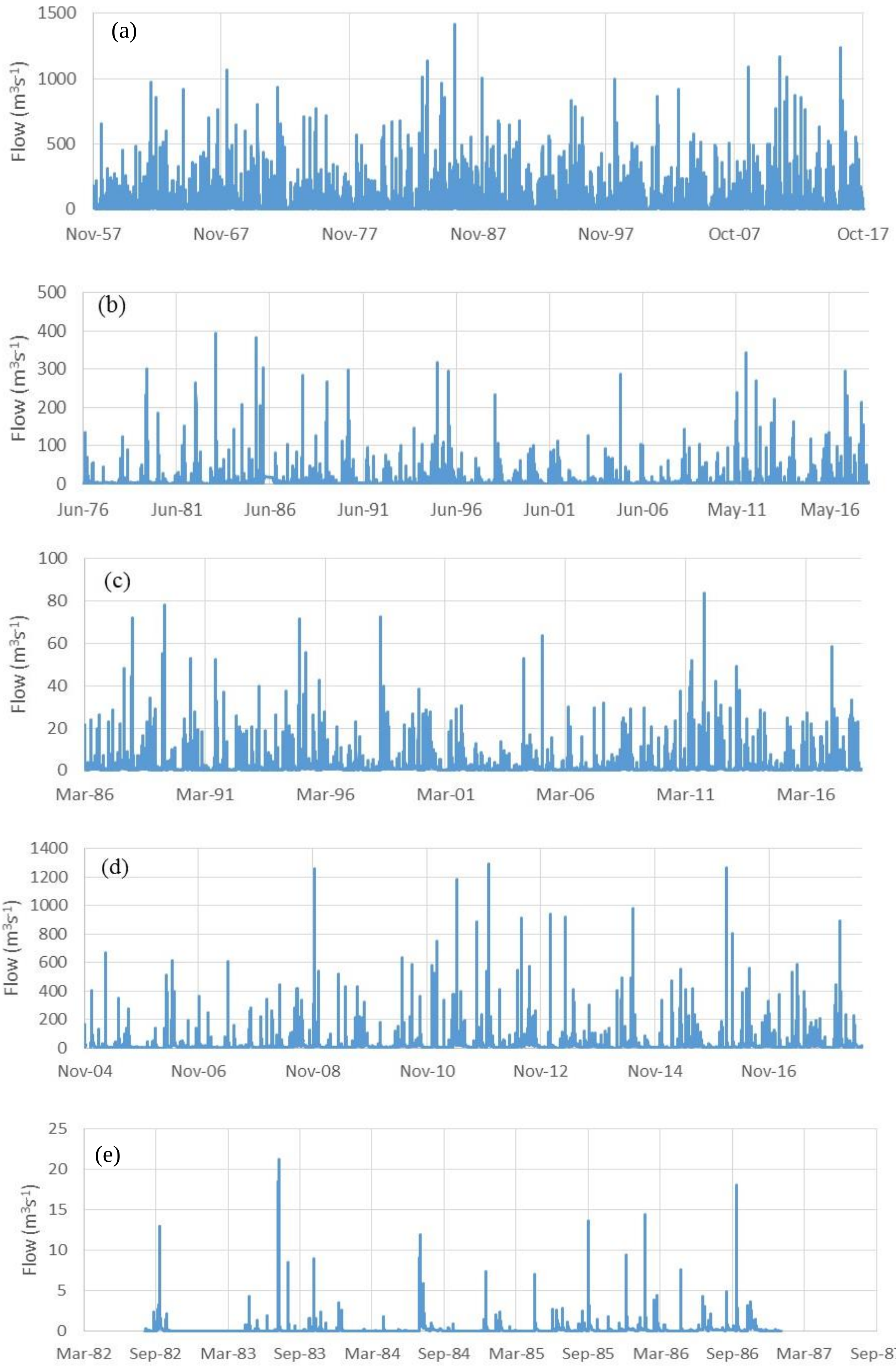


Figure 6.3.Flow over for time: (a) Wairoa Combined gauge (b) Wai-iti Combined gauge (b) Wai-iti at Belgrove gauge (d) Waimea at TDC Nursery gauge € Pitfure at Johnston's gauge

To record stage, and therefore flow, accurately, a gauge is ideally fixed at a cross-section that is deep and narrow. The locations of the gauges in this study were mostly at gorges and bridges. Here the cross-sections were generally well-defined channels where flow was unlikely to bypass the gauge, allowing the gauge to capture the entirety of the stage and flow.

Preliminary models showed no significant backflow within the model. This was confirmed by the TDC who communicated that there was no backflow upstream of the Appleby Bridge.

The 1983 flood event was used for calibration, however, there was no available gauge data at the lower boundary condition (TDC Nursey) or upper boundary condition (Wai-iti at Belgrove gauge). Because of this a rating curve was constructed for the lower boundary condition using historic data from the downstream gauge. For the missing upstream boundary condition the flow was approximated using the TM61 method.

The gauges recorded data every 15 minutes. The flows were linearly interpolated to 30-second intervals. This ensured that the boundary conditions did not experience relatively large increases and decreases in flow over a single time step, this increased the stability and realism of the model.

6.2.2. Ungauged Catchments

Along the Wai-iti River there were several ungauged tributaries, because these tributaries contributed the majority of the flow they were significant and therefore had to have their flows estimated. During the December 2011 flood the Wai-iti at Belgrove gauge recorded a maximum flow of 84 m^3s^{-1}. Further downstream the Wai-iti Combined gauge recorded a flow of 344 m^3s^{-1}. This left 260 $m^3s^{-1}$from ungauged catchments. The transposition of gauged streamflow from nearby catchments to an ungauged catchment is called regionalisation. Regionalisation is widely regarded as a difficult challenge in hydrological science (Oudin, Andréassian, Perrin, Michel, & Moine, 2008; Sivapalan et al., 2003), because of the lack of data to calibrate model parameters to. Studies on regionalisation methods usually produce differing results as they have examined different sites, and also, because the available catchment characteristics vary from case to case (Razavi & Coulibaly, 2013). As a result, there is no single widely accepted method for regionalisation. In New Zealand however there are a number of recognised methods for estimating ungauged catchment flows. Each method was used to approximate the ungauged peak flowrates during the 1983 event. These methods were used to calculate the flows and are outlined in section 4.3. The shape of the ungauged hydrograph was based off the shape of the Pitfure stream hydrograph recorded for the event. This was because the Pitfure stream was the closest spatially and in terms of catchment size. The areas of the catchments were defined by watershed boundaries provided by the TDC.

The regional method is not applicable to catchments less than 10 km^2. Because of this the method could not be applied to the Hoult or Teapot valley catchments. Because the catchments tended to not fall on the contour curves an approximation was used to determine the contour values. This introduced uncertainty in the estimation; this coupled with the 22% uncertainty in the contour maps resulted in significant uncertainty regarding the peak flow values.

The modified rational method required the slope of the ungauged catchments to be calculated. This was achieved using the equal area method. Using the 25m DEM of New Zealand provided by Land Resource Information Systems (Landcare Research, 2010), a terrain plot of each catchment was created. The equal area method calculates the slope using a hypothetical line, which is positioned so that the enclosed areas above and below it are equal. An example plot is shown in Figure 6.4.

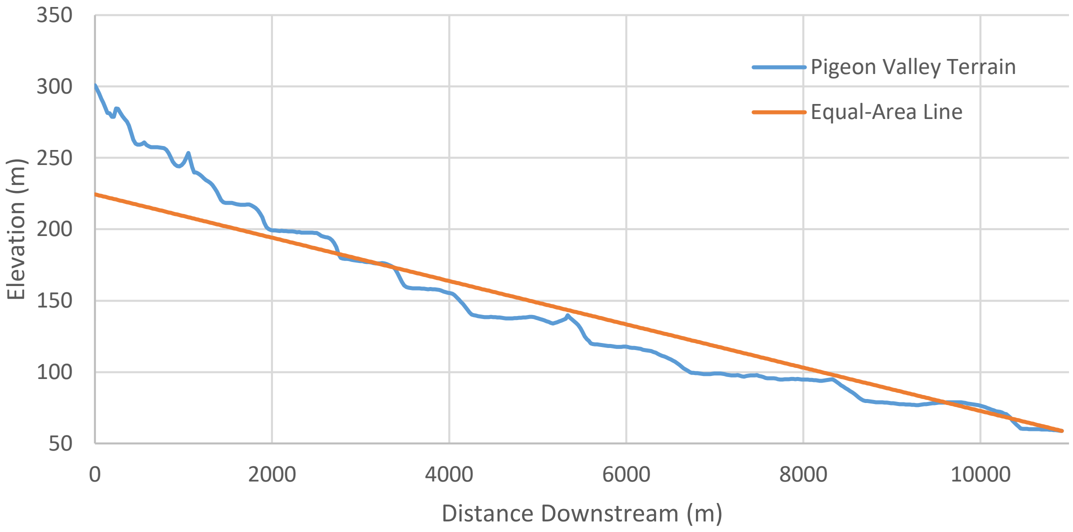


Figure 6.4. Equal-area method for Pigeon Valley example

To find the runoff coefficient for the modified rational method, the land cover of each ungauged catchment area was extracted from the LCDB (Landcare Research, 2015a) and the percentage area of each type of terrain calculated. This data was used to interpolate a runoff catchment that was more representative of the whole catchment. The ungauged catchments averaged about 87% bush. Most of this was made up of Exotic forest.

The rainfall factor required an estimation of the rainfall intensity and a return period to be specified. The rainfall intensity for this method was calculated from data provided by High-Intensity Rainfall Design System (HIRDS) a tool provided by NIWA (NIWA, 2018b). HIRDS can be used to estimate high-intensity design rainfall depths at any point in New Zealand. It was designed to be used for assessing storm rarity and hydrological design purposes. The Wai-iti Combined gauge was used to specify the return period of the event as it has the longest record of flows on the Wai-iti River and there gave a more accurate estimation of the actual return period. The Wai-iti Combined gauge (as of the 6th July 2018) had recorded the 1983 event as a 25.2-year event. It was assumed that the return period was the same for all the ungauged catchments. This return period was used in the HIRDS tool to derive the rainfall intensity.

For the TM61 method, the slopes and the percentage of bush cover, found in the modified rational method, were used to calculate the coefficient C.

The rainfall factor was calculated from the design rainfall depth and the standard rainfall depth. These depths depended on the time of concentration. The time of concentration was the time taken a unit of water to travel from the furthest point in the catchment to the catchment outlet. TM61 provided three different methods for this but warned against using the average time and recommended selecting the most reasonable time for a catchment. The time of concentration for the study area was calculated using all three methods. Two of the methods gave times that correlated to an average velocity of $1.5 ms^{-1}$, this was deemed too fast to be realistic. The Bransby-Williams Method (Wanielista, Kersten, & Ealgin, 1997) was selected as the time corresponded to a velocity of approximately $0.75\ ms^{-1}$. This velocity was deemed more sensible and was used in the calculations for intensity. Using the time of concentration and the return period (given by the Wai-iti Combined gauge) in conjunction with the data from HIRDS, the design rainfall depth was found.

The standard rainfall depth was found using the standard rainfall curve provided in TM61; this also depended on the time of concentration. Using the design rainfall and standard rainfall depths the rainfall factor was calculated for each ungauged catchment.

The shape factor took into account the total area of the catchment as well as the direct length from the furthest point in the catchment to the outlet. These lengths were found using ArcMap, as was the catchment area.

Following the calculation of the flood peaks in all the ungauged catchments for the 1983 event, the methods were compared. A comparison of the methods against the observed data and the total flow of all the ungauged catchments is shown in Table 6.2.

Table 6.2. Comparison of peak flow methods

	Pitfure Peak Flow (m^3s^{-1})	Pitfure Peak Flow Percentage	Total Flow of all catchments (m^3s^{-1})	Total Flow of all catchments difference
Observed Data	21.3	-	398	-
Scaled Method	21.3[1]	0	362	91%
Regional Method	32.6	153%	449	113%
Modified Rational Method	24.7	116%	430	108%
TM 61	24.2	114%	404	102%

1) *Scaled method used the flow data from the Pitfure gauge to recreate the ungauged catchment flows. This resulted in the estimation of the Pitfure flow being the same as the gauge data.*

The TM61 method had the closest peak flowrate to the Council derived data at the Pitfure and Wai-iti Combined gauges. There was total flow peak difference of 6 m^3s^{-1}; this was sufficiently accurate to recreate the ungauged flows. The method was also independent from the gauged data, unlike the scaled method. This made the method appropriate to use for flow validation purposes at the Wai-iti Combined gauge.

The TM61 method was also the most rigorous method considered, accounting for the most catchment characteristics out of the methods used. The characteristics considered included: land cover, average channel slope, channel length, rainfall intensity and duration, catchment area, and the shape of the catchment. For these reasons, the TM61 method was selected to recreate the hydrographs for the 1983 event.

It was assumed that the return period was the same in all the catchments and that the event started in all catchments at the same time

6.2.3. Design Hydrographs

To assess the impact of the undocumented stopbanks, a range of return periods were assessed that required design hydrographs to be created. The peak flows for the design hydrographs were created by performing a Generalized Extreme Value Analysis (GEV) on the annual maximum flow levels for the Wairoa and Wai-iti Combined gauge, records of 62 and 43 years respectively. For the Wai-iti River: the location, scale, shape parameters of the GEV analysis were 133, 76 and -0.059 respectively. These had 95% confidence intervals (CI) of ±29, ±22 and ±0.374 (22%, 29%, and 633%). The diagnostic plots of the GEV analysis are shown in Figure 6.5.

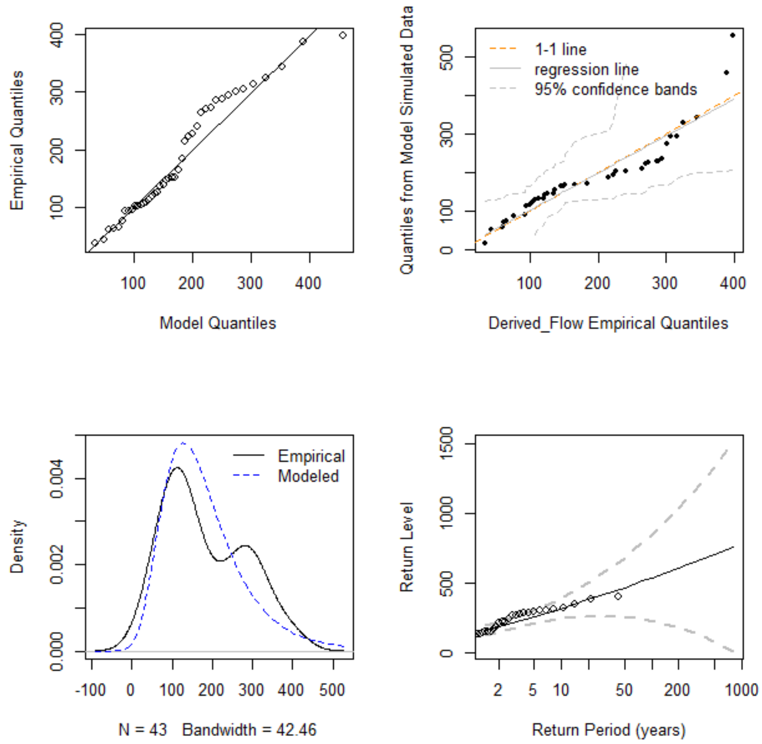


Figure 6.5. GEV analysis of flowrate diagnostic plots

A GEV analysis also allowed the uncertainty to be considered for each return period as shown in the lower left diagnostic plot. The CIs became wider as the return period increased as there were fewer observed data points. Beyond a return period of 100 years, the CI became impractically large for flow estimation purposes. Given the dual-peak nature of the Council derived flow data this was a reasonable fit. From the GEV estimations, the flow for a range of return periods was calculated, as depicted in Table 6.3.

Table 6.3. GEV estimate of flowrate with confidence intervals

Estimations of peak flow at the Wai-iti Combined Gauge			
Return Interval	95% lower CI (m^3s^{-1})	Estimate (m^3s^{-1})	95% Upper CI (m^3s^{-1})
5-year	207	253	298
10-year	246	317	387
20-year	263	381	498
50-year	255	468	680
100-year	227	536	846

The peak flow for the design event was distributed between the gauged and ungauged catchments based on the average percentage of flow contribution. Along the Wai-iti, the TM61 method allowed the expected flow for the ungauged catchments to be calculated. These flows were found as a percentage of the sum of these ungauged and gauged flow for each of the calibration and validation events. This percentage corresponded to the expected percentage contribution to the total flow. The expected percentage contribution was calculated and then averaged for all the calibration and validation events to find the average expected percentage contribution. For each catchment, the percentage of flow contribution for all event were with 2% of each other. The average percentage of flow contribution for each catchment is shown in Table 6.4.

Table 6.4. Percentage of flow contribution for each catchment in each of the events simulated

Percentage of flow contribution (%)					
Catchment	1982 Event	1983 Event	1986 Event	2011 Event	Average Percentage of Contribution
Pitfure Catchment	6.1	5.2	6.1	6.0	**5.8**
Teapot Valley Catchment	7.1	6.2	7.1	7.1	**6.9**
Pigeon Valley Catchment	13.2	13.7	13.2	13.2	**13.4**
Hoult Valley Catchment	4.5	4.3	4.5	4.5	**4.5**
Trass Valley Catchment	5.4	4.6	5.4	5.4	**5.2**
Pretty Bridge Valley Catchment	11.8	11.4	11.8	11.8	**11.7**
Eighty Eight Valley Catchment	11.0	11.4	11.0	11.0	**11.1**
Wai-iti Catchment	27.5	29.3	27.5	27.6	**28.0**
Quail Valley Catchment	13.4	13.8	13.4	13.4	**13.5**
TOTAL	**100**	**100**	**100**	**100**	**100**

The shape of the design hydrographs were based on historic flood hydrographs. For the Wai-iti and Waimea catchments there were gauges that had recorded the hydrograph shape for several large flood events. For these catchments, the unit hydrographs for the five largest events on record were calculated. The peak flows of these unit hydrographs were aligned and averaged to find the design unit hydrograph, seen in Figure 6.6.

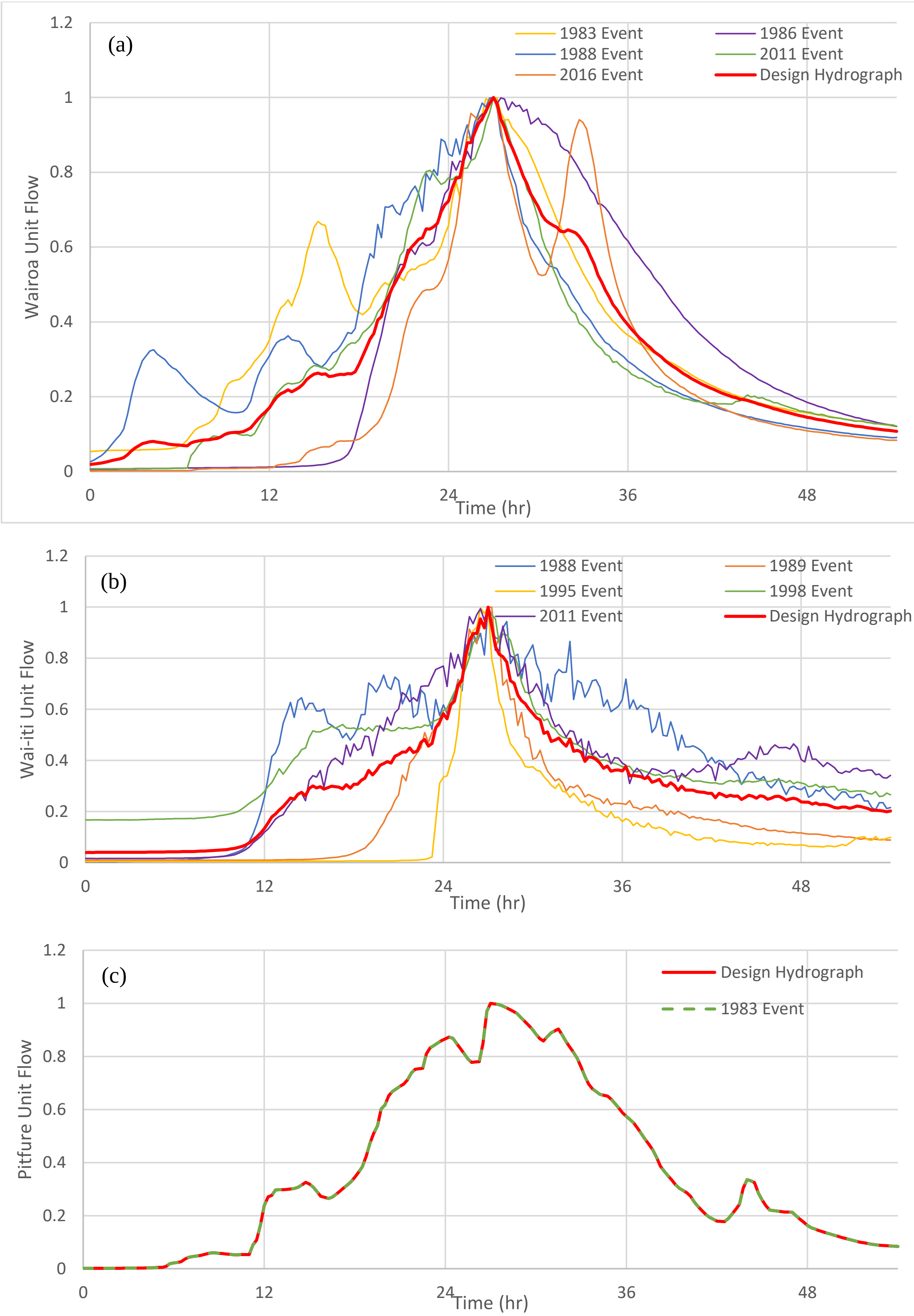


Figure 6.6. Design hydrographs for: (a) the Wairoa catchment (b) the Wai-iti catchment (c) the Pitfure and ungauged catchments

As the Pitfure gauge only had five years' worth of data there was a limited number of large events to define the shape of the unit hydrograph from. Because of this, the shape of the Pitfure hydrograph was based only on the 1983 event, as this was the largest event recorded.

The shape of the design hydrographs for the ungauged catchments were the same as the Pitfure catchment as it was the closest gauged catchment in terms of location, size, and flow contribution.

6.3. Terrain

Accurate topographical data have been shown to be a crucial determining factor in the accurate prediction of flood inundation (FEMA2009). Critical topographic features such as stopbanks can be relatively narrow structures, yet they have a significant impact on flood routing. It was therefore important that the topographical data fully captured the height and extent of the stopbanks.

6.3.1. LiDAR

The topographical data were supplied by the Council as two Digital Elevation Models (DEM) created from Light Detection and Ranging (LiDAR). The LiDAR utilised an airborne laser system to illuminate terrain with a pulsed laser light; by measuring the laser return time and differences in wavelength a three dimensional representation of the terrain was created.

Because the laser reflects off the first object encountered, thick vegetation along the river banks may have interfered with the accuracy in some areas; however, the LiDAR data were captured with 2.0 points per m^2, meaning the last-return was likely to be of the ground through gaps in the vegetation (AAM, 2017). Major bridges in the DEM were also removed during the LiDAR post-processing

6.3.2. Bathymetry

Because the LiDAR could not penetrate below the surface of the water it could not capture the bathymetry. To incorporate the bathymetry, cross-sections taken by the TDC in 2016 were used along the Wai-iti and Wairoa Rivers. The final DEM used LiDAR on land as it did not require additional interpolation, unlike the cross-section data. Cross-section data was used in the channels as it more accurately represented the bathymetry, shown in Figure 6.7. Where cross-sections were not available the cross-sections were interpolated within HEC-RAS.

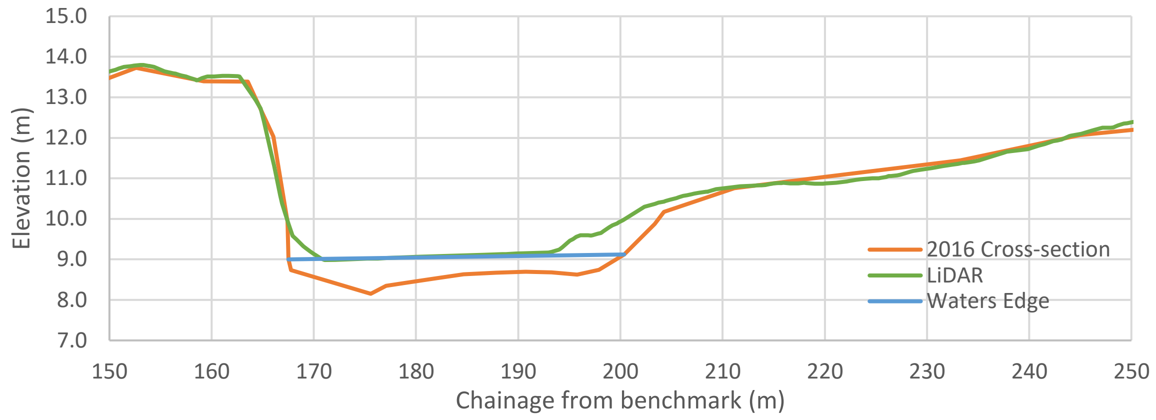


Figure 6.7. Typical Waimea River cross-section

The cross-sections taken used the Nelson Vertical Datum 1955 (NVD55) and the Council DEMs used the New Zealand Vertical Datum (NZVD2016). The adjustment between the two datums' varied spatially. To make the cross-section datum compatible, the benchmark heights used in the cross-sections were found in NZVD2016. From this, the differences in height were found and averaged at each cross-section. The average difference was then applied across the cross-section; this ranged between 0.26 m and 0.36 m.

The channel bed was misaligned in some locations due to the meandering nature of the Wai-iti and Wairoa Rivers. The survey and LiDAR data were also taken at separate times. To correct this the cross-section in some locations was readjusted so the cross-section and LiDAR channels aligned.

Within the study area there were six smaller bridges along the Pitfure that were present in the LiDAR data. Within the model the sides of these bridge extended into the channel, preventing flow from passing underneath and causing a blockage. Because the Pitfure Stopbank was being analysed it was important the flow was accurately represented. Similarly, in heavily forested areas the trees registered as the ground surface in the LiDAR and blocked the flow. At these locations, the artificial blockages were removed and the terrain height interpolated.

6.3.3. Land cover

Within HEC-RAS different land covers were specified to describe different roughness values that increased and decreased the depths and velocities of flooding. The New Zealand Land Cover Data Base (LCDB) (Landcare Research, 2015a)has classified the land cover for all of New Zealand. Using the LCDB to define the model land cover there were 18 unique classes within the study area. At the upper Wai-iti, the most prominent land covering was 'high producing exotic grassland'. Downstream of the confluence point the area of 'orchard, vineyard or another perennial crop' and 'short-rotation crop' increased. The relevant definitions of the land covers can be seen in Table 6.5. Based on the classifications, a Manning's n value was applied to each cover type, this created a more realistic model with multiple roughness values.

The initial values for the land cover Manning's roughness were found from literature and are detailed below in Table 6.5 (Chaudhry, 2008; French, 1985; Munson, Young, & Okiishi, 2010; Streeter, Wylie, & Bedford, 1998).

Table 6.5. LCDB 4.1 relevant land cover definitions (Landcare Research, 2015b) and literature land cover Manning's n values

LCDB Classification	Practical Interpretation	Description	Manning's n	Domain Area (%)
Built-up Area (settlement)	Township	Commercial, industrial or residential buildings, including associated infrastructure and amenities, not resolvable as other classes. Low density 'lifestyle' residential areas are included where hard surfaces, landscaping and gardens dominate other land covers.	0.03	1%
Gravel or Rock	River channel	Bare surfaces dominated by unconsolidated or consolidated materials generally coarser than coarse gravel (60mm). Typically mapped along rocky seashores and rivers, sub-alpine and alpine areas, scree slopes and erosion pavements.	0.033	18.6%
Short-rotation Cropland	Production land	Land regularly cultivated for the production of cereal, root, and seed crops, hops, vegetables, strawberries and field nurseries, often including intervening grassland, fallow land, and other covers not delineated separately.	0.075	3.7%
Orchards, Vineyards or Other Perennial Crops	Orchards and vineyards	Land managed for the production of grapes, pip, citrus and stone fruit, nuts, olives, berries, kiwifruit, and other perennial crops. Cultivation for crop renewal is infrequent and irregular but is sometimes practised for weed control.	0.035	12.7%
High Producing Exotic Grassland	Production land	Exotic sward grassland of good pastoral quality and vigour reflecting relatively high soil fertility and intensive grazing management. Clover species, ryegrass and cocksfoot dominate with lucerne and plantain locally important, but also including lower-producing grasses exhibiting vigour in areas of good soil moisture and fertility.	0.035	56.4%
Exotic Forest	Native forest	Planted or naturalised forest predominantly of radiata pine but including other pine species, Douglas fir, cypress, larch, acacia and eucalypts. Production forestry is the main land use in this class with minor areas devoted to mass-movement erosion-control and other areas of naturalised (wildling) establishment.	0.15	1.5%
Indigenous Forest	Commercial forest	Tall forest dominated by indigenous conifer, broadleaved or beech species.	0.15	2.0%

Originally the LCDB defined the Wairoa, Wai-iti and Waimea River channels under the classification ‘rock or gravel’ and neglected the tributaries. As the Pitfure and other tributaries had rock or gravel channel beds, the extent of the ‘gravel or rock’ land cover was increased to encompass the tributary streams. Without this increase, the channel roughness of the Pitfure stream bed and other tributaries could not be individually calibrated. A friction map is shown in Figure 6.8.

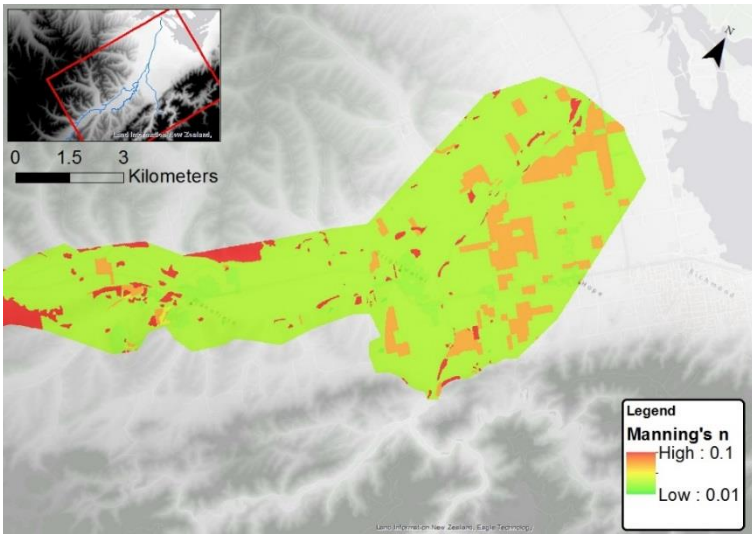


Figure 6.8. Initial Manning's n values within the study area

The classifications for the LCDB were last updated in 2012. The model, therefore, used land cover definitions that were seven years out of date.

Examination of land covers from 1996, 2001, 2008 and 2012 showed that the area of ‘Built-up area (settlement)’ in the study area did not increase significantly over a 16 year period. townships tend to be the most valuable land cover per unit area and if flooded have the largest cost and impact on society. Because the area of townships was not significantly increasing it was deemed acceptable to use the 2012 land covers to simulate the flood events.

7. CALIBRATION AND VALIDATION

7.1. Model Stability

An appropriate cell size was essential to develop a model that gave numerically stable results that made physical sense whilst also having an acceptable computational time. Preliminary modelling found that the majority of the channel and floodplain had velocities less than 5.0 and 1.0 ms^{-1} respectively.

Using the Courant equation a selection of viable computation time intervals and cells sizes were created and compared. It was impractical to have a model with a computation time longer than a day. However, as the mesh became larger the model was not always able to recognise smaller features within the terrain such as banks and roads. This lead to water 'leaking' through features that should have prevented flow and an inaccurate result.

The Courant number for the 12 m mesh in the channel was generally below 0.8 and 0.2 for the majority of the channel and the floodplain respectively. Although the mesh did experience some 'leaking', the differences in flood extent around the stopbanks were minimal, which was the key focus of this study. In HECRAS breaklines align cell faces to a line defined by a user. Breaklines were used in areas prone to 'leaking' to reduce its effects by forcing the model to capture the terrain.

7.2. Model Calibration

The calibration process involved optimizing channel and floodplain friction coefficients in order to increase the model accuracy, which was assessed against observed inundation extent. The primary means of calibrating was through predicting inundation extent for historical flood events and comparing with observed maps of inundation extent.

In ideal conditions there would be flood extent maps for multiple independent events during periods where data are also available for all domain gauges. Some of these could be used for calibration and others for validation, to provide a robust model verification process. For this research, the TDC provided four historical flood maps of the study area, for events in 1982, 1983, 1986, and 2011. However, only the 1983 flood map covered the entire extent of the model domain and, because of this, was used to calibrate the model, with other historical maps being used for validation.

The calibration process involved optimizing the Manning's n values. The calibration took place in two stages. The first stage focused on calibrating the floodplain. The second stage focused on calibrating the roughness values of the Wai-iti and Wairoa channels.

As previously mentioned there were 18 land covers in the domain, 17 relating to land usage and one that was only found in river channels. Optimising 18 land covers would have been excessive and for many land covers redundant, as they were not expected to be inundated during the simulation. Because of this, during the first phase, the dimensionality was reduced to two parameters: channel friction and floodplain friction values, which were increased from their initial values using a universal scaling.

The success of the model's parameters was quantified using an F-value which compared the historical and modelled inundation extents. This was used as the primary means of calibrating the model. The method for this is outlined in section 4.4.1.

The optimum floodplain Manning’s values were +75% above the literature values and gave an F-value of 0.62 when the channel roughness was 0.04. In comparison, when the floodplain Manning’s value was lowered to -30% from the literature values the F-value was 0.59. Conversely, when the Manning’s values were increased by +350% of the literature values the F-value was 0.60. The optimisation of the floodplain was not significant in terms of model performance, seen in Figure 7.1, as a significant increase/decrease in Manning’s n values did not significantly increase/decreased the F-value. This shows the model was not sensitive to the floodplain roughness values.

During the first phase of calibration, it was found the model was sensitive to the Manning’s value of the channel, seen in Figure 7.1. The optimal Manning’s n value for the channels was 0.040. When the floodplain and channel were at their optimum values the F-value was 0.6188.

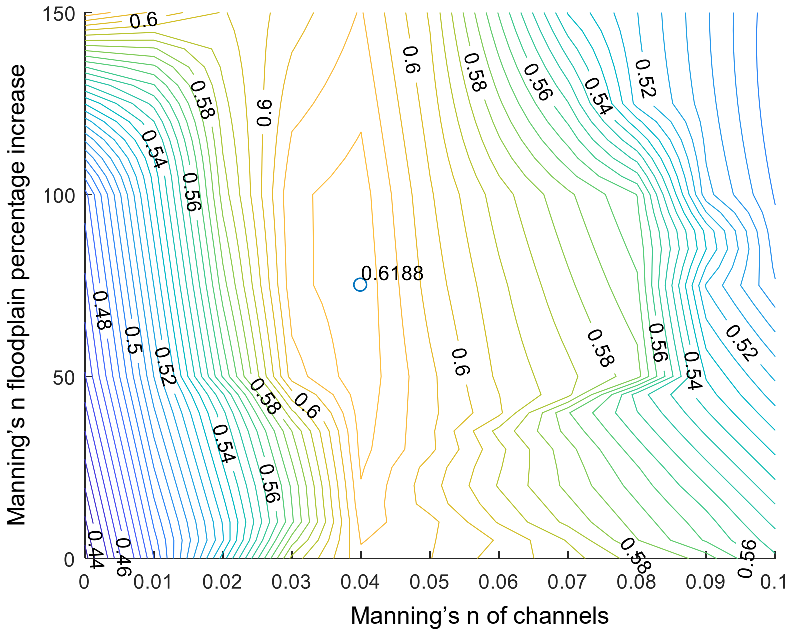


Figure 7.1. F-value contour plot of Manning floodplain percentage and channel bed value

The second phase of calibration focused on optimising the individual river channels because the Waimea reach was over predicting the inundation extent and the Wai-iti reach was under predicting, shown in Figure 7.2. This made physical sense as the characteristics of the channels were different. The average discharge and average slope were 14 m^3s^{-1} and 0.003 for the Wairoa River, and 2.0 m^3s^{-1} and

0.005 for the Wai-iti River. The calibration of the individual channel improved the ability of the model to prediction inundation extent.

To calibrate the channel friction, the different channel beds were spatially divided into two sections, the Wai-iti (that encompassed the Pitfure and the ungauged catchments) and the Wairoa (that also included the Waimea). During this process the floodplain Manning’s n values were held constant at their optimal values.

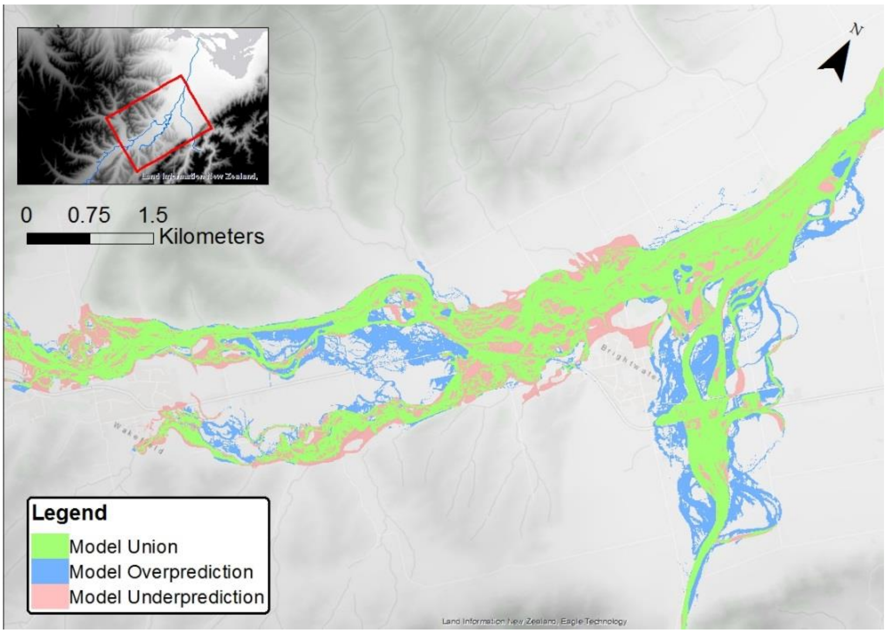


Figure 7.2. Model performance flood map with floodplain +75% and channel roughness equal to 0.04

The calibration process of the second phase followed the same process as first phase. Figure 7.3 shows the calibration curves of the F-value for the Wai-iti and Wairoa Manning’s channel values.

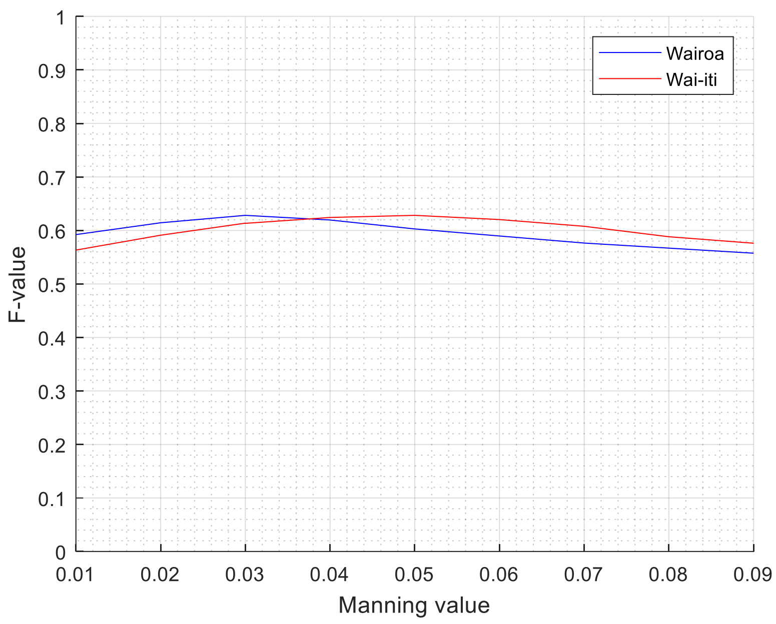


Figure 7.3. Calibration curve of the Wairoa and Wai-iti Manning channel values

The amount of overlap (union), underprediction, and overprediction are calculated as percentages of the historical inundation area.

The rate of change in the calibration curve decreased at higher Manning's n values because the rate of change in overlap (union) and underprediction decreased, making overprediction the primary driver for change in the F-value. The reason for this was because there were preferential pathways for flow in the model terrain. This meant that certain areas (that were flooded in the historical map) were unlikely to be flooded in the model. This caused the overlap and underprediction areas to not increase/decrease as rapidly at higher Manning's n values, as shown in Figure 7.4. Mathematically, the percentage of underprediction is equal to one hundred minus the percentage area of overlap. Because the overlap area was not increasing, the underprediction was also not decreasing.

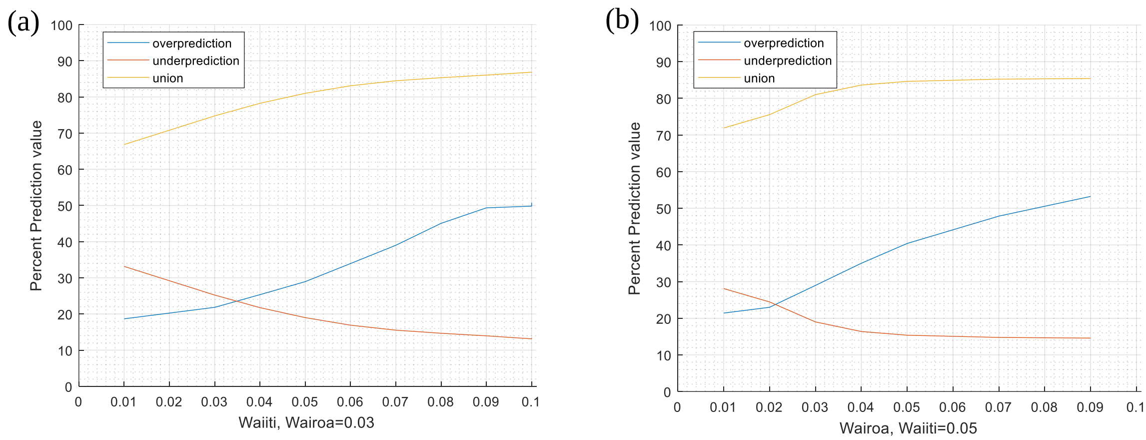


Figure 7.4. Calibration curve components for the: (a) Wai-iti channel roughness (b) Wairoa channel roughness

The optimum Manning's n values were 0.05 and 0.03 for the Wai-iti and Waimea respectively. These Manning's n values were within the range of the literature values for similar rivers (0.026 to 0.074 and 0.025 to 0.042 for the Wai-iti and Wairoa respectively) (Hicks & Mason, 1991).

The individual calibration of the channels caused a slight improvement in F-value. With all the optimised parameters the F-value was 0.63 (an improvement of 0.01). The inundation extent of the calibrated model after phase 1 and phase 2 can be seen in Figure 7.5.

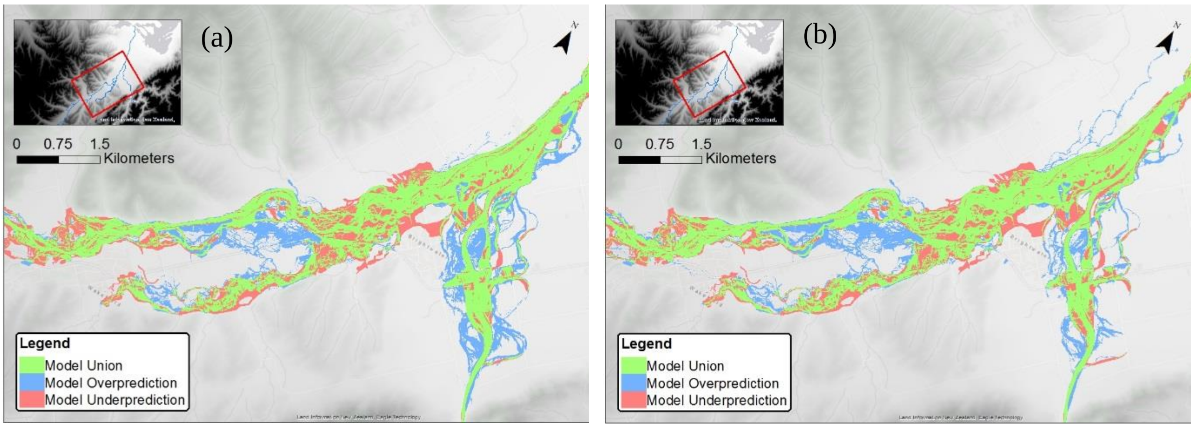


Figure 7.5. Model performance with: (a) floodplain = +75% and channel roughness = 0.04 (b) floodplain =+75%, Wai-iti channel roughness = 0.05, and Wairoa channel roughness = 0.03

The model was more sensitive to the Wai-iti channel value than the Wairoa (Figure 7.6). This was logical as Wai-iti flooded a larger area and was therefore more influential.

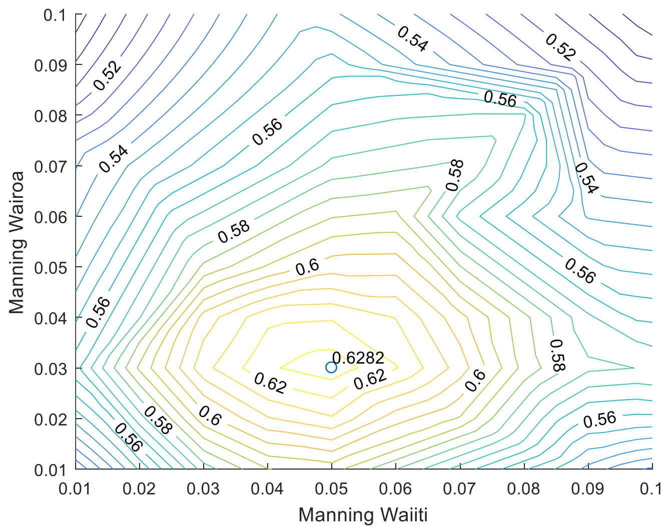


Figure 7.6. F-value contour plot of Wai-iti and Wairoa Manning channel value

The differences in flood extent were most likely due to terrain differences. The model used 2016 data which could be different from the terrain in 1983. Gravel extraction began in the Waimea after the 1983 event, and the degradation from this is thought to have propagated upstream to the Wai-iti. A comparison of river cross-sections from 1999 and 2016 showed that in some locations the bed has dropped up to one metre, as shown in Figure 7.7. This view is supported by observations from landowners within the area, "*The riverbed is lower now, than it was then & the river seldom floods across to the bank*" (Higgins, 2018).

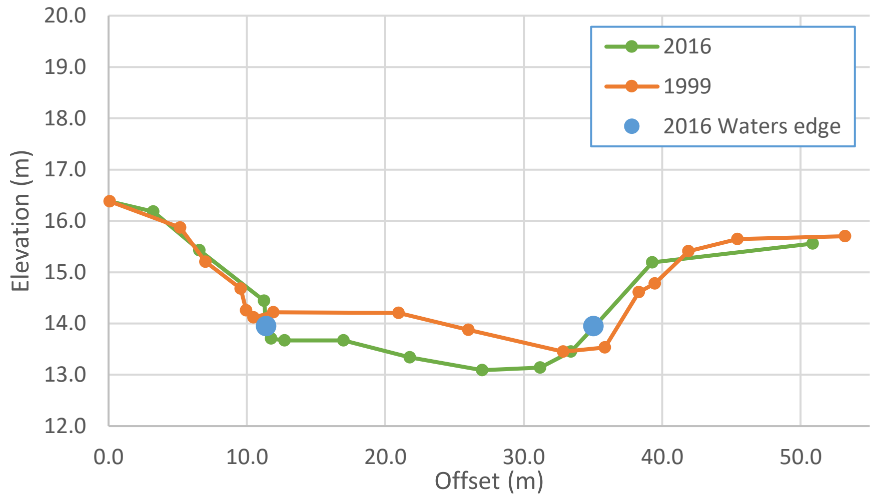


Figure 7.7. Comparison of the cross-section at Wai-iti Combined gauge between 1999 and 2016

7.3. Model Validation

Three separate events were used to validate the model. This allowed the accuracy of the calibrated model to be tested and ensure that the roughness values were not overfitted to the 1983 event.

The range of validation events was limited by the availability of the flood maps. Other than the July 1983 event there was no single map that covered the entire model domain. Some of the gauges for the validation events were missing data; in these cases, the flow was recreated using the TM61 method. The validation inundation extents can be seen in Figure 7.8.

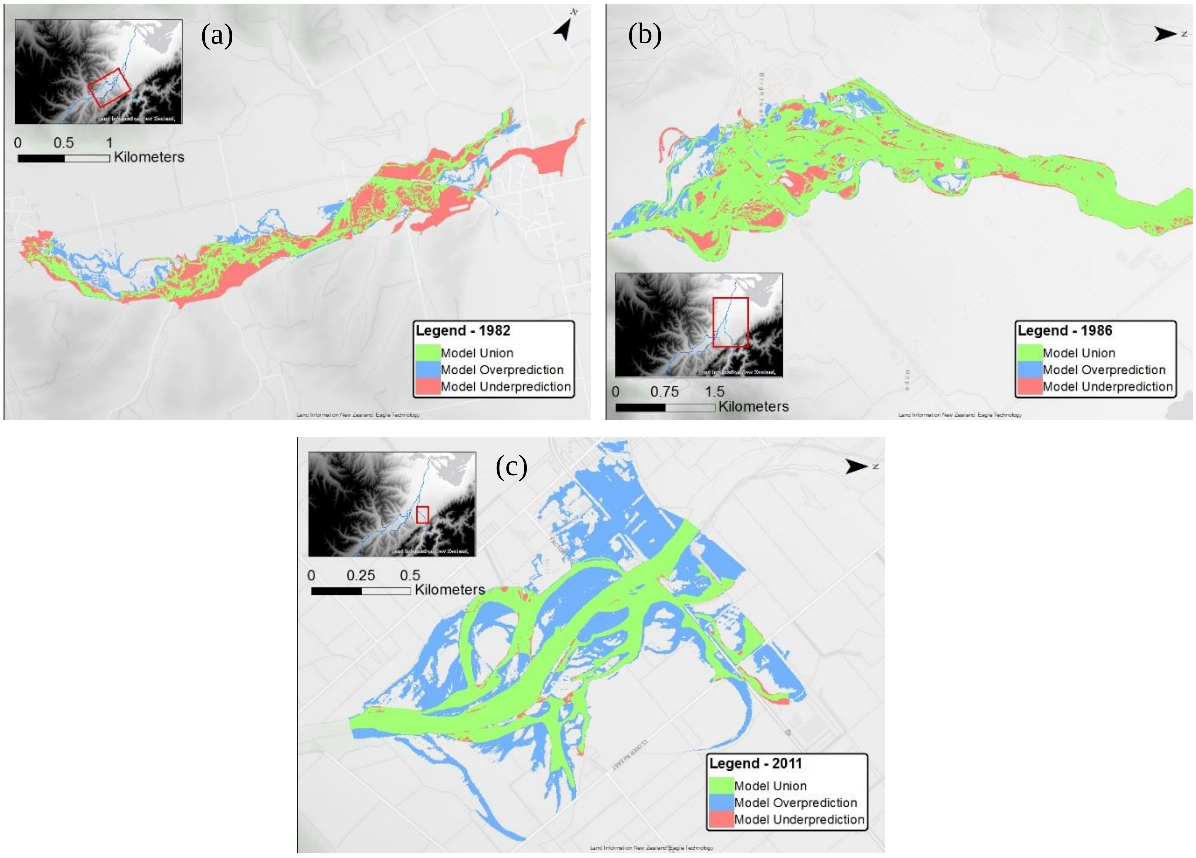


Figure 7.8. Validation flood maps of the: (a) 1982 event (b) 1986 event (c) 2011 event

The F-value validation results are shown below in Table 7.1.

Table 7.1. Validation F-value results

Event	Area of flood map	Gauge data at:	F-value
2011 May	Wairoa	Wairoa, Wai-iti Combined, Wai-iti Belgrove	0.3859
1986 Jan	Wairoa, Waimea	Wairoa, Wai-iti Combined, Pitfure	0.7998
1982 Jun	Pitfure	Wairoa, Wai-iti Combined	0.4489

The 1982 flood map only covered the Pitfure Stream. The model recreated this event relatively well. The calibrated model overprediction, underprediction, and overlap areas relative to the historical flood extent were 23%, 45%, and 55% respectively. The high underprediction may have been related to new infrastructure in the model, such as raised roads which were not present in the 1983 terrain. Raised roads such as State Highway 6 (SH6) protected some areas from flooding. Other roads also had this effect. The Wai-iti River had the most underprediction. A further reason for the high underprediction was that the flow in the model may not have accurately represented the actual flows. There was only one gauge along the Wai-iti which was at the downstream end. Because of this much of the data had to be estimated and likely resulted in differences at the Pitfure. Although the flow hydrographs from the Council derived and modelled flows were similar, the storm may have been more concentrated around the Pitfure catchment and led to higher flows and more inundation around the Pitfure than modelled.

The flood map for the 1986 event covered the Wairoa and Waimea Rivers. The model performed well at recreating this event. The calibrated model overprediction, underprediction, and overlap areas relative to the historical flood extent were 13%, 9%, 91% respectively. The high F-value was most likely due to a significant part of the flood map being constrained between the Council stopbanks. This made a high overlap easier to achieve and due to the increased area made differences elsewhere less significant.

The reason the calibrated model performed poorly in the 2011 event was that the model overpredicted significantly with 56% more area flooded than the historic flood map. This may have been because the model used 2016 LiDAR data which had been calibrated to a 1983 flood event.

Overall the F-values indicated that the model worked adequately to recreate historical flood events. There were differences between modelled inundation extents and historic flood maps, however these were not unreasonable and were explainable.

7.4. Nash-Sutcliffe

The Nash-Sutcliffe value was used to quantify the accuracy of model stage heights and flow rates. The method used for calculating the Nash-Sutcliffe value is shown in section 4.4.2. The Nash-Sutcliffe value was calculated from the historic Wai-iti Combined gauge and compared against the modelled data. The following Nash-Sutcliffe value analysis was not used for calibration or validation and should be considered supplementary to the F-value analyses, it provides an insight into how the model flows and depths compare to those observed. The stage difference and flow graphs for the calibration and validation event are shown in Figure 7.9. The Nash-Sutcliffe values for the model are shown in Table 7.2. The Nash-Sutcliffe values were considered reasonable given the methods and assumptions used to recreate the ungauged catchment flow rates.

In all cases, the model overpredicted the maximum stage difference, by 0.62 m, 1.32 m, 0.97 m, and 1.86 m for the 1983, 1982, 1986, and 2011 events respectively. The Nash-Sutcliffe values indicate that the model preformed poorly at recreating the observed stages, however the model was calibrated to predict the inundation extent and not depth.

Gravel extraction was likely to have caused channel degradation and the bed to drop. As the model was calibrated to match the 1983 inundation extent using the 2016 channel depth it was understandable that the modelled stage was deeper as it had to account for the lower bed. This was a possible explanation for the overprediction of channel water depth

Table 7.2. Nash-Sutcliffe validation results

	NASH-SUTCLIFFE VALUE			
Event	**1983**	**2011**	**1986**	**1982**
Flow	0.79	0.80	0.58	0.94
Stage	0.48	-1.43	-1.33	-0.57

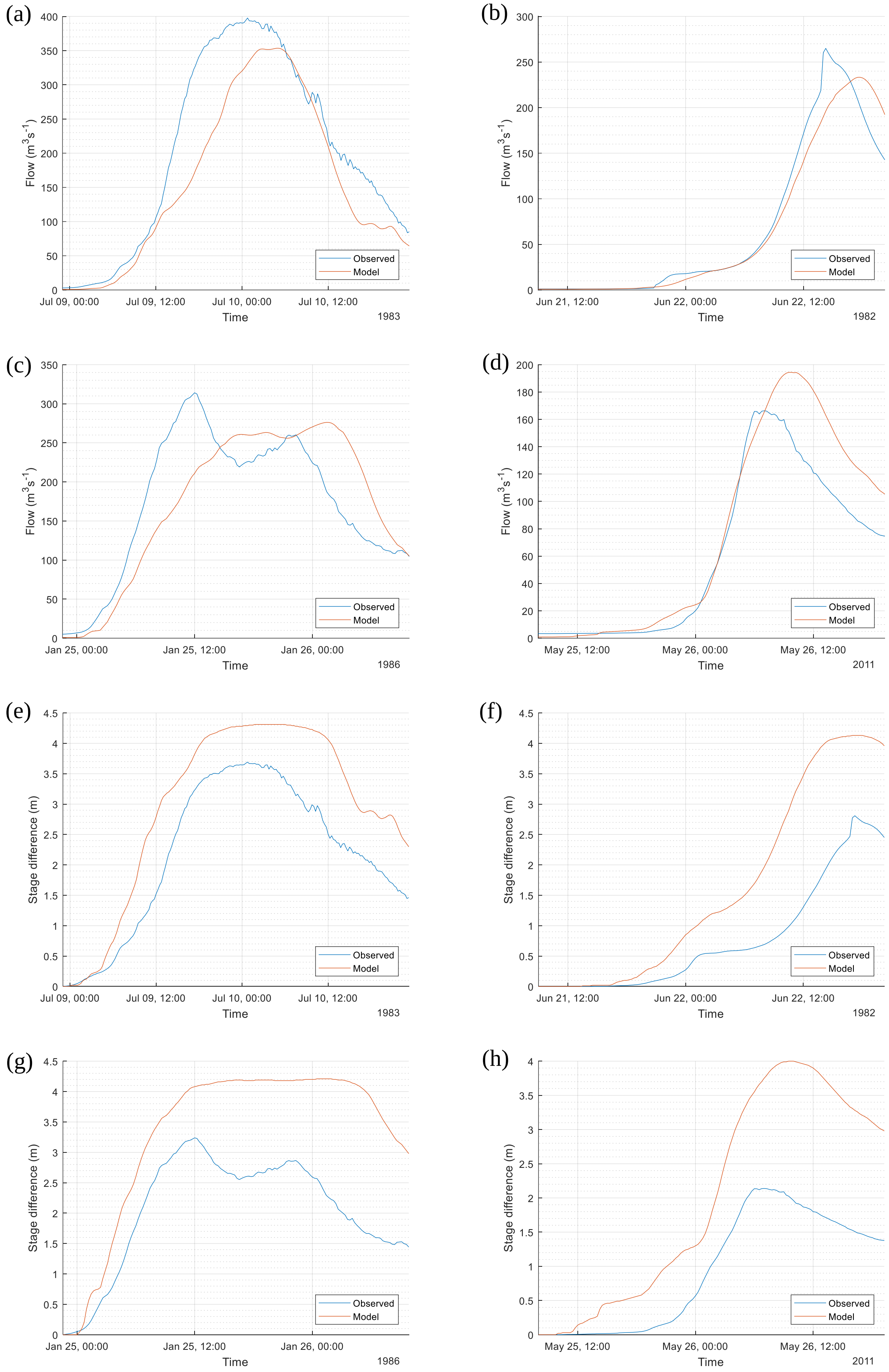


Figure 7.9. Validation graphs for: (a) 1983 event flow (b) 1982 event flow (c) 1986 event flow (d) 2011 event flow (e) 1983 event stage difference (f) 1982 event stage difference (g) 1986 event stage difference (h) 2011 event stage difference

The flowrates of the models tended to be slower to rise than the Council derived data. This was most likely a result of the way the ungauged catchment hydrographs were delayed. The ungauged hydrographs were delayed based on the Bransby-Williams time of concentration. This assumed that the ungauged catchment areas receive rain at the same time, and that there is no spatial variation in rainfall intensity. This assumption gave a more attenuated hydrograph downstream and higher Nash-Sutcliffe value than if it was assumed all the catchments began to flood at the same time. As a result of delaying the hydrographs the flood peaks were also delayed. At the Wai-iti Combined gauge, the delays in peak flow for the 1983, 2011, 1986, and 1982 floods were 4.25, 2.5, 13.5, and 3.5 hours respectively. The Nash-Sutcliffe value for each of the events was acceptable. The 13-hour delay for the 1986 event was caused by the two peak formation of the hydrograph, the delay between the first peaks of the hydrographs was actually 4.75 hours.

The volume of rainwater was not accounted for in the model. It is probable that if the model had included the rain volume it would have led to a steeper rise in the modelled hydrographs and a closer match to the Council derived data. During the historical event, rainfall may also have been distributed further downstream than modelled which would have caused a shorter travel time and an earlier flood peak.

7.5. Summary

The 1983 flood event was used to calibrate the model as it was the only event that had a flood map of the entire model domain. The model was calibrated by comparing the historic and modelled inundation extent using the F-value statistic.

By grouping the floodplain Manning's values, the dimensionality was initially reduced to two parameters. From this, the floodplain roughness was calibrated and optimised. Following this, the floodplain roughnesses were held constant and the roughness of the Wai-iti and Wairoa channels were individually calibrated. The model was more sensitive to the channel roughness values than the floodplain roughness as it carried the majority of the water. The model was verified against three different flood maps from 1982, 1986, and 2011, each of these maps inundated of different areas of the domain.

The Nash-Sutcliffe values were calculated for the flow and stage of the validation events to give an indication of their accuracy. These values for the flows were greater than zero, meaning that they were a better approximation than simply using the mean flow. The F-values for the validation events ranged between 0.38 and 0.8. The most likely reason for these values was that the model was calibrated to an 1980's flooding extent while using 2016 topography. Overall the model performed acceptably well to assess the impact of the undocumented stopbanks.

8. FLOOD EXTENT ASSESSMENT

To assess the impact of the undocumented stopbanks, a range of flood design hydrographs events were simulated in a range of stopbank scenarios. The method to obtain the design hydrographs for the flood events is shown in section 6.2.3. The flood events simulated had return periods of 5, 10, 20, 50, and 100 years. The scenarios assessed the implications of removing and maintaining the documented and undocumented stopbanks. This determined the benefits and potential consequences of the stopbanks with relation to inundation extent. This information can be used to make more informed decisions about flood risk management.

The models for all of the simulated scenarios used the same parameters such as flow inputs, mesh size, and time interval. This terrain used the 2016 LiDAR data on land and 2016 cross-section data for the bathymetry. The only difference between scenarios were that the Council and undocumented stopbanks geometry were modified to simulate their removal/maintenance. This method ensured differences arose solely from the removal/maintenance of the stopbanks.

The first scenario represented the current situation with all stopbanks present (D1U1). The removal of all of the stopbanks (D0U0) was simulated and represented the worst-case scenario. The removal scenarios (MSG0, Pit0, Con0) involved the individual removal of stopbanks to isolate their impacts on flood extent.

From the inundation extents of these scenarios it was apparent that the undocumented stopbanks were non-uniform which allowed overtopping in several regions. The maintained scenarios (MSGR, PitR) investigated the implications of better maintaining the undocumented stopbanks. Only these stopbanks were further investigated as these had the most noticeable impact on flooding extent of the undocumented stopbanks. The methods used to develop the maintained scenario are outlined in section 4.5. The maintained scenarios represented what would happen if the undocumented stopbanks were uniform and aimed to determine the benefit in increasing the protection they provided.

The scenarios have been labelled according to Table 8.1. Sections 8.1 to Section 8.8 discuss the implications of these scenarios in terms of locations and reasons for flooding as well as the land uses flooded and how these change as the return period increases.

Table 8.1. Scenarios and definitions

Scenario	Definition	Documented	MSG	Pitfure	Confluence
D1U1	All documented and undocumented stopbanks present	✓	✓	✓	✓
D0U0	All documented and undocumented stopbanks removed	✗	✗	✗	✗
D1U0	All stopbanks present except the documented stopbanks are removed	✓	✗	✗	✗
MSG0	All stopbanks present except the MSG stopbank is removed	✓	✗	✓	✓
Pit0	All stopbanks present except the Pitfure stopbank is removed	✓	✓	✗	✓
Con0	All stopbanks present except the Confluence stopbank is removed	✓	✓	✓	✗
MSGR	All stopbanks present and the MSG stopbank is maintained	✓	✓✓	✓	✓
PitR	All stopbanks present and the Pitfure stopbank is maintained	✓	✓	✓✓	✓

8.1. All Stopbanks Present - D1U1

The purpose of this scenario was to give an overview of the current flood hazard and to provide a control scenario to compare against other scenarios. This allowed the benefits of each scenario to be assessed. An overview of the inundation extent is shown in Figure 8.1.

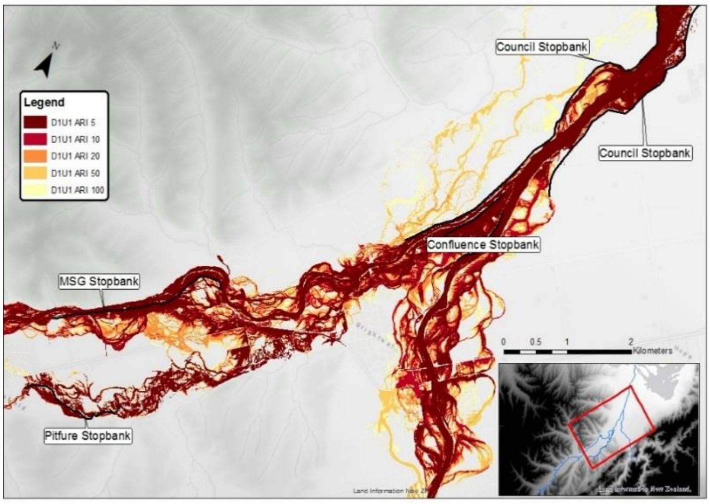


Figure 8.1. Overview of D1U1 flooding extent

In a 5-year event, the Council stopbanks contained the flood while maintaining a large amount of freeboard. Further upstream, the Pitfure stopbank had a fluctuating crest height. Consequently, flood flows were able to overtop parts of the stopbank and inundate the adjacent production land. There were eight areas where water overtopped the Pitfure stopbank and at two of these water flowed back into the channel. Figure 8.2 shows the extent of flooding along the Pitfure stream when the stopbank is present.

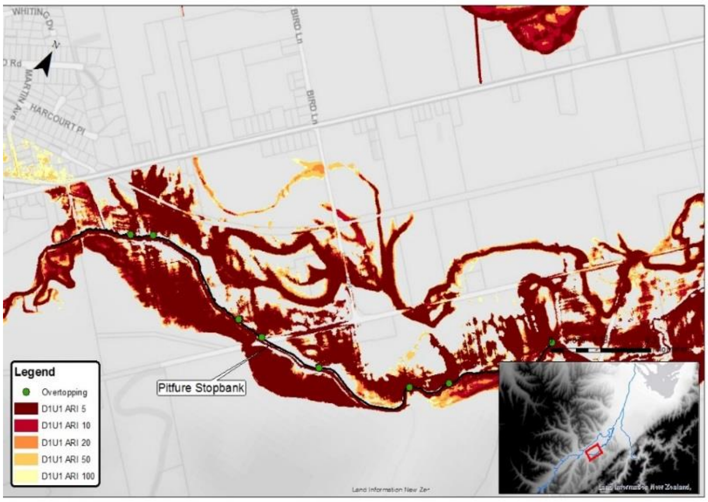


Figure 8.2. D1U1 flood extent at the Pitfure Stream

The MSG stopbank overtopped in six locations during the 5-year event which allowed water into the adjacent production land. Similar to the Pitfure, overtopping occurred in areas where the surface of the stopbank crest dropped. Along the MSG stopbank, the overtopping occurred in areas with impenetrable cover. Due to the nature of LiDAR, the accuracy of topographic data decreases as the amount of vegetation increases because fewer points are able to strike the ground.

This may have contributed to the formation of the undulations in the DEM, which resulted in the overtopping. The areas of overtopping are highlighted in Figure 8.3. The locations of overtopping are generally in alignment with areas of deterioration observed during the condition assessment.

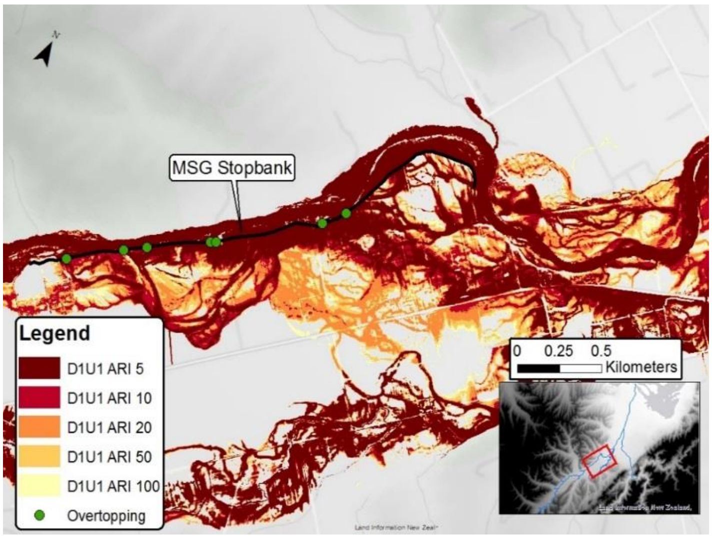


Figure 8.3. D1U1 flood extent at the MSG stopbank

In this study, erosion of stopbanks was not considered. The stopbanks were considered fixed, permanent structures and assumed to be indestructible. Given the velocities involved in the overtopping of the MSG stopbank (up to of 2.0 ms^{-1}), there is evidence to suggest the stopbanks would be eroded to some extent in the event of a significant flood (Hewlett, Boorman, & Bramley, 1987).

In total, the 5-year event inundated a total area of 8.1 km^2. This is likely an overprediction of the extent as the model does not account for any infiltration which ccould reduce the inundation. The soil within the floodplain has been classified as 'well drained' within New Zealands soil map (Manaaki Whenua, 2019). Table 8.2 categorised the area flooded by the land cover as defined by the 2012 LCDB definitions. It was apparent from the table that the majority of the inundated land was used for horticultural or agricultural purposes.

Table 8.2.Land cover of inundated areas

2012 LCDB classification	Area (km^2)	Area Percentage (%)
Production land	4.44	54.9
River bed	1.93	23.8
Orchard/Vineyard	0.86	10.6
Forestry	0.18	2.2
Short-rotation Cropland	0.17	2.2
Native Forest	0.15	1.9
Exotic Forest	0.13	1.6
Township	0.02	0.3
TOTAL	8.10	

The 10-year event inundated an additional 2.4 km^2 compared to the 5-year event. The additional flooding was generally evenly distributed throughout the domain. Notably, along the Wairoa River, the flow was able to overtop the river bank near the Fonterra Brightwater building.

The 20-year event continued the trend of inundating more area. The MSG stopbank overtopped in eleven locations and these were all at the upstream end of the stopbank at depressions in crest height. The result of the overtopping was that more production land was flooded and flow from the Wai-iti joined the Pitfure Stream further upstream. This connection did not significantly increase the inundation area along the Pitfure Stream.

During the 20-year event, water flowed around the upstream end of the Council stopbank on the Wai-iti side. During the 50-year event, this flow increased and joined the Eves Valley Stream. Also during the 50-year event, flow from the Wairoa River exited the channel upstream and flooded areas of Brightwater Township. At the downstream boundary, the vast majority of the volume was contained by the Council stopbanks. The 100-year event flooded more area than the other return periods.

A time series of the inundated area is presented below in Figure 8.4. It should be noted the peak inundation area and maximum inundation area were different. The peak inundation area was the maximum area inundated a single time. The maximum inundation area was the total of area inundated at any time during the simulation.

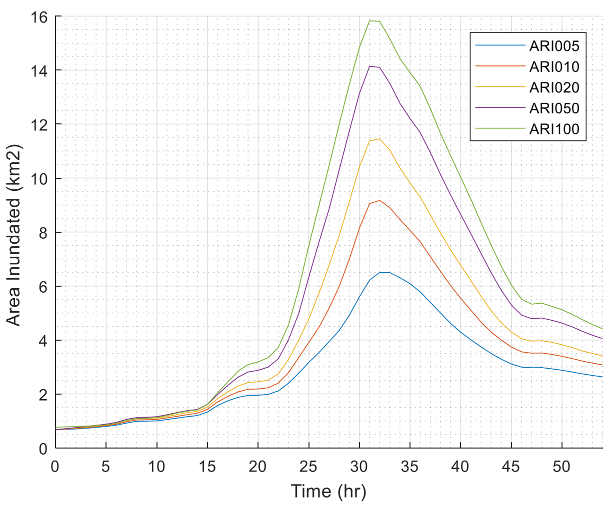


Figure 8.4. D1U1 area inundation against time

Where present, the Council stopbanks were able to constrain the 100-year flood with at least one metre of freeboard. However, the water was able to exit the channel upstream of the Wai-iti side of the Council stopbank. This caused inundation west of the Council stopbank. Because of the Pitfure stopbanks undulating surface, water was allowed into the adjacent land. The MSG stopbank overtopped in some areas where there were to be undulations in the DEM. In these areas there were brambles and impenetrable cover.

8.2. No Stopbanks Present - D0U0

This scenario represented all stopbanks being removed and replaced with natural terrain. The 5-year event demonstrated that the stopbanks prevented significant inundation of areas west of the stopbank. In the D1U1 scenario, flow only reached Eves Valley Stream during events greater than 50-years, and in this case the inundation was relatively minor. The absence of the stopbanks allowed water to flood significant areas of land (4.1 km^2 in a 5-year event), which is shown in Figure 8.5, comparing a 5-year event with and without stopbanks. Flooding also occurred along the east Council stopbank footprint at the downstream reach of the domain. The land that was additionally flooded was a mixture of production land and orchards/vineyards.

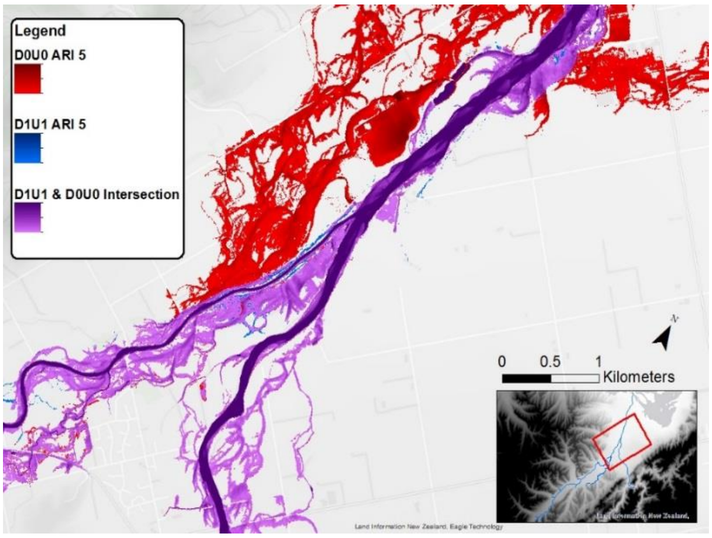


Figure 8.5. Flood extents of D1U1 and D0U0 in a 5-year event at the Council stopbanks

The lower domain had a zero stage boundary condition outside the main Waimea channel added to allow water to flow out of the domain and prevent water building up in the lower parts of the domain. However, as a consequence of this, backflow effects were not present outside of the channel.

At the Pitfure, the removal of the stopbanks caused one additional point of overtopping that led to more volume exiting the channel compared to the D1U1 scenario. Because of this, there was a very slight increase in area inundated; this is shown in Figure 8.7.

Along the Wai-iti River, the removal of the stopbanks allowed additional flow to cross into the adjacent production land. The flow rate over the MSG stopbank footprint is depicted in Figure 8.6. This shows that the removal of the undocumented stopbanks allowed double the amount of flow, and 2.75 times more volume (1,765,000 m^3) to travel across the MSG stopbank footprint. The increased volume inundated areas around Southfuels Spring Grove petrol station and along Barton lane before joining the Pitfure Stream.

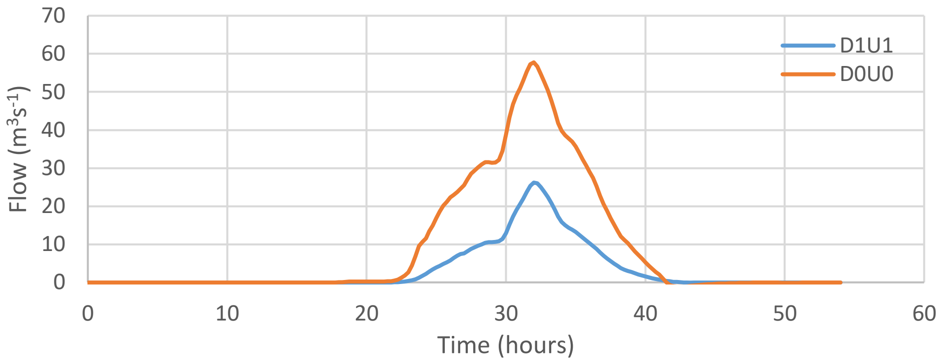


Figure 8.6. Flowrates in a 5-year event across MSG stopbank

The difference in the inundation area at the Pitfure and MSG stopbanks is shown in Figure 8.7. As a consequence of the extra flow joining the Pitfure, there was less flow in the Wai-iti River (15 m^3s^{-1} at peak flow). Because of this, the area (that was flooded in the D1U1 scenario) at the end of the MSG stopbank was not flooded. This area appears to be an airstrip.

Investigation of the flow paths found that there was no backflow in this area and that all the flow inundating this area was from the Wai-iti River. Therefore, this inundation was only due to the presence of the MSG stopbank. In other words, the presence of the MSG stopbank caused this airstrip to become inundated up to a depth of 0.2 m.

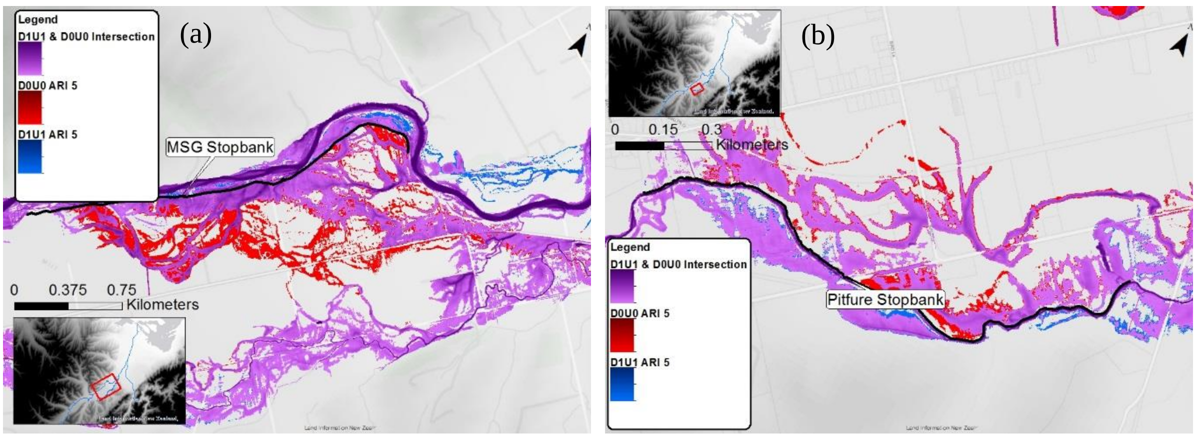


Figure 8.7. Flood extent of the D1U1 and D0U0 in a 5-year event at the: (a) MSG stopbank (b) Pitfure stopbank

As the return period increased, the differences between the D1U1 and D0U0 scenarios diminished along the Wai-iti River and Pitfure Stream. At the Council stopbanks, the difference became more apparent as increased flow inundated greater areas of the adjacent land.

From Figure 8.8 combined with Figure 8.4, a 20-year event with all the stopbanks removed inundated more area than a 100-year event when all the stopbanks were present. The difference in total inundation area during a 5-year event was 4.1 km^2, this area was mostly located adjacent to the Council stopbanks.

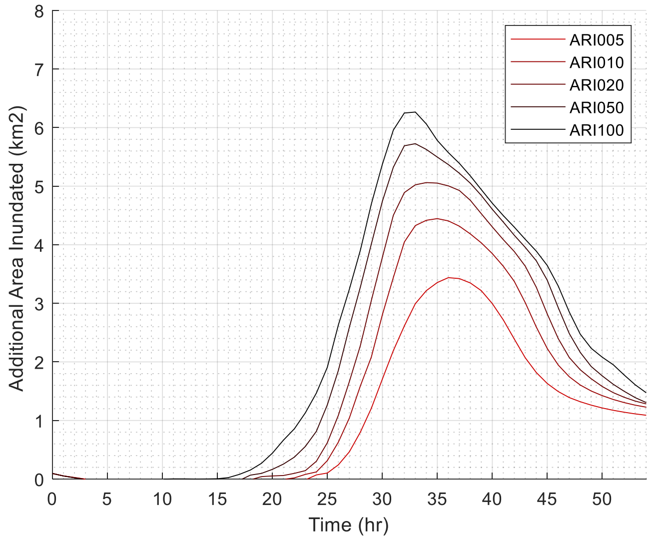


Figure 8.8. Additional area inundated against time (D0U0-D1U1)

8.3. No Undocumented Stopbanks Present - D1U0

This scenario highlighted the impact of the undocumented stopbanks as a collective by comparing the inundation extent to the D1U1 scenario. Comparison of this scenario with the D0U0 scenario highlighted the effect of the Council stopbanks. The flooding extents are shown in Figure 8.9.

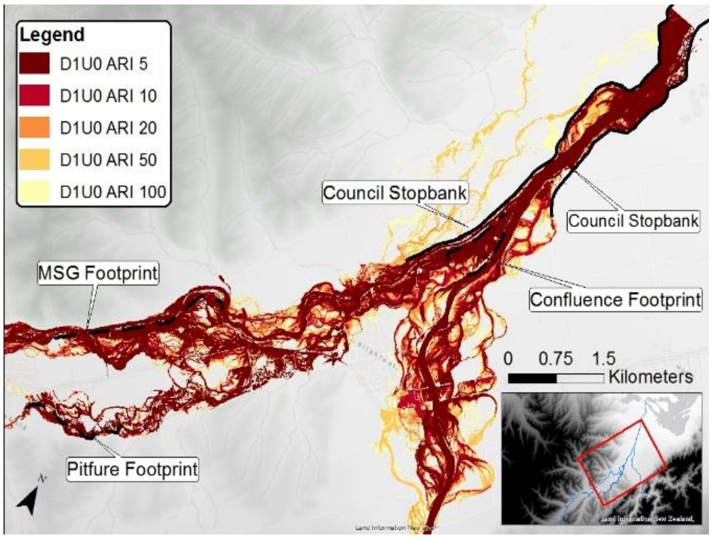


Figure 8.9. Overview of D1U0 flooding extent

Within the model, the removal of the undocumented stopbanks did not significantly increase the flooding extent because in the D1U1 scenario they overtopped. The confluence also experienced overtopping, however, the main source of flow into the confluence area was the flow around the top end of the stopbank. This flow originated from water exiting the Wai-iti channel near Waimea W Bridge. The extent of this overtopping and flow around the undocumented stopbanks is examined further in the following sections.

The flooding extent upstream was very similar to the D0U0 scenario. This indicated that constraining the flow to between the Council stopbanks caused minimal backwater effects. However, at the Confluence stopbank, in a 5-year event, there was more inundation. The additional area inundated is shown in Figure 8.10. The additional inundation was most likely because the Council stopbanks contained more flow which resulted in additional water entering the confluence area. This effect was examined in the Con0 scenario.

The difference in inundation extent between the D1U0 and D1U1 in a 5-year event was 0.53 km^2. The difference in inundation extent between the D0U0 and D1U0 scenarios in a 5-year event was 3.7 km^2. This illustrates the importance of the Council stopbanks, as they prevented significantly more area from being inundated than the undocumented stopbanks.

There were two notable areas where the removal of the Council stopbanks allowed a significant area to become inundated. During a 5-year event, the presence of the Council stopbanks prevented flow passing across the footprint at the upper end of the Wai-iti. In the D0U0 scenario, this flow tended to follow historic stream beds before joining Eves Valley Stream. The Council stopbanks also prevented flow exiting the channel on the true right on the lower end of the stopbanks. The flow from this ran across the SH60 until it reached the model boundary.

The flood map of a 5-year event is shown in Figure 8.10, comparing the downstream extents of the D0U0 and D1U0 scenarios. From this, it was apparent that the Council stopbanks played an important role in reducing the flooding extent of the Waimea River and Wai-iti River.

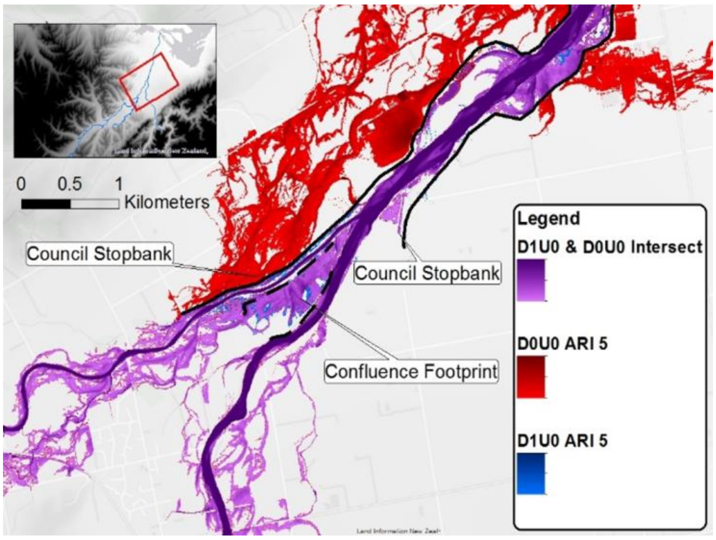


Figure 8.10. Flood extents of D1U0 and D0U0 in a 5-year event at the Council stopbanks

8.4. Main Spring Grove Stopbank Removed - MSG0

This scenario simulated the current terrain except the MSG stopbank was removed. This isolated the impact of the MSG stopbank which allowed a more comprehensive analysis of the impacts of the MSG stopbank to be conducted in comparison to other scenarios.

The 5-year event found that, in comparison to the D1U1 scenario, there was little increase in inundation extent. In the land adjacent to the MSG stopbank, the flooding extent was identical to the D0U0 scenario. Water flowed across SH60 with a depth of 0.2 m near Spring Grove and joined the Pitfure Stream. The different inundation extents are shown in Figure 8.11.

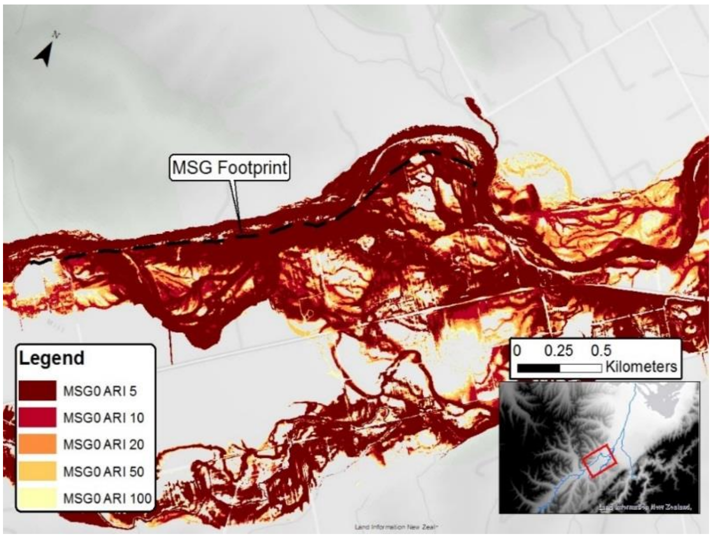


Figure 8.11. Overview of MSG0 flooding extent

In the D1U1 scenario, water overtopping the MSG stopbank flowed into the Pitfure. In the MSG0 scenario this happened four hours earlier because greater flows were able to inundate the adjacent land and join the Pitfure. This is shown in Figure 8.12. The lack of the MSG stopbank also allowed more volume to enter the Pitfure Stream (217,000 m^3s^{-1}). Neither of these led to significantly more area along the Pitfure Stream becoming inundated compared to the D1U1 scenario. The removal of the MSG stopbank also reduced the inundation extent at the airstrip in a 5-year event. Although this reduction was not large, it demonstrated that the MSG stopbank, by reducing the inundation in one area could inundate a different area downstream. The differences in inundation areas are illustrated in Figure 8.13.

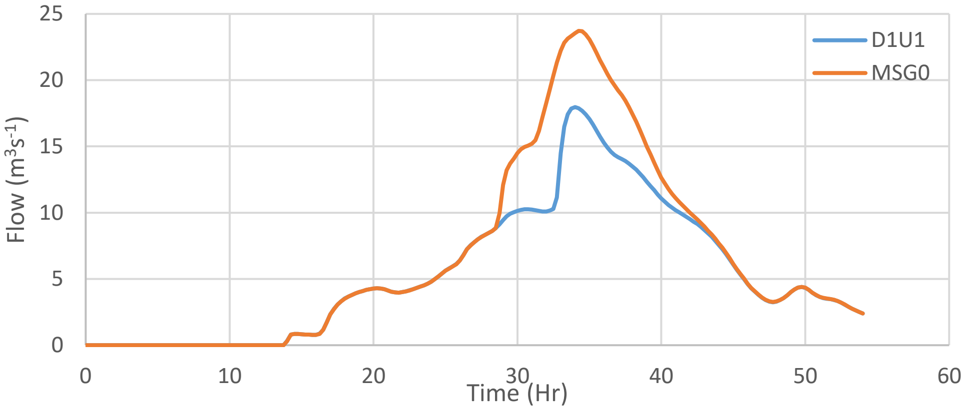


Figure 8.12. Flowrates in a 5-year event adjacent to the at Roughton Lane

Overall, the presence of the MSG stopbank prevented 0.43 km^2 of land from inundation in a 5-year event. The additional area was predominately production land. The difference in area flooded is shown in Figure 8.13, comparing D1U1 and MSG0 in a 5-year event.

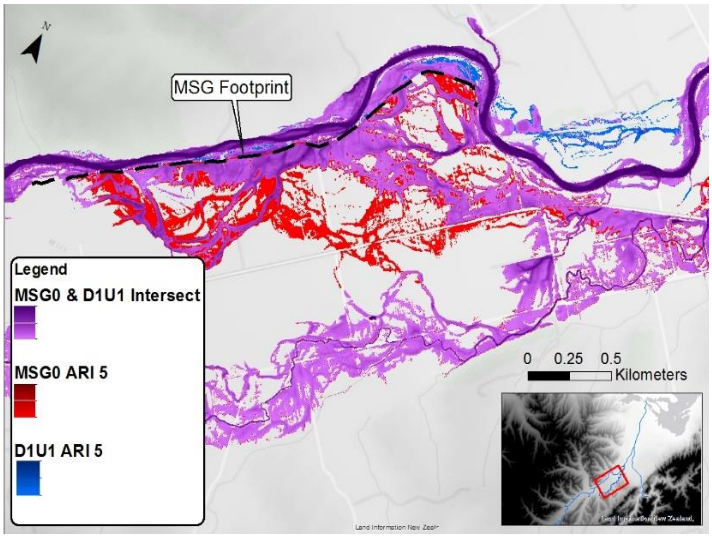


Figure 8.13. Flood extents of D1U1 and MSG0 in a 5-year event

As the return period increased, the area of inundation also increased; however the difference in inundation area decreased between the D1U1 and MSG0 scenarios. This showed that the protection provided by the MSG stopbank decreased in larger events. The depths were also similar in these larger events. The difference in flooding is shown in Figure 8.14.

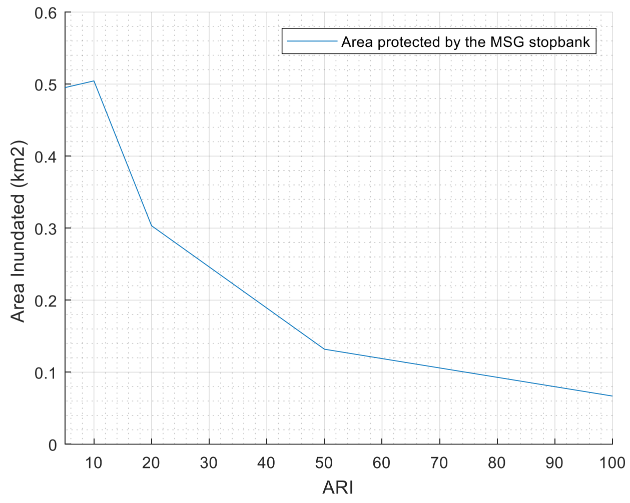


Figure 8.14. The difference in inundation extent caused by the MSG stopbank

8.5. Pitfure Stopbank Removed - Pit0

This scenario simulated the current terrain, except the Pitfure stopbank was removed. This allowed the impact of the Pitfure stopbank to be independently assessed. The inundation extents for the different return periods are shown in Figure 8.15. A comparison of the Pit0 and D1U1 scenarios in a 5-year return period is also presented in Figure 8.15.

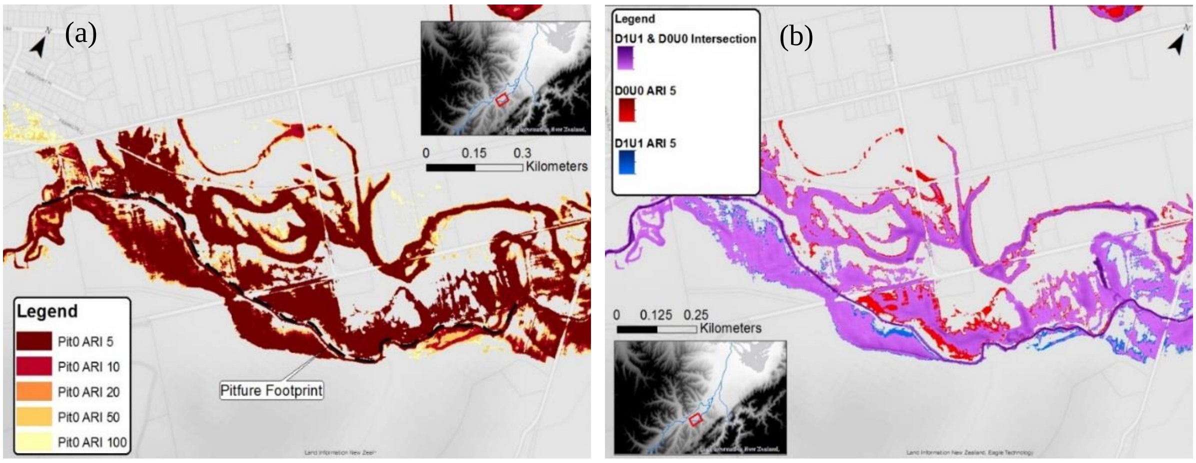


Figure 8.15. (a) Overview of Pit0 flooding extent (b)Flood extents of D1U1 and Pit0 in a 5-year event

The removal of the Pitfure allowed slightly more flow into the adjacent production land. During a 5-year event this did not cause significantly more area to become inundated (0.094 km^2), nor did it cause the depths to increase significantly. The average depth increased in the adjacent land by 2 cm (maximum average depths of 0.18 m and 0.20 m, with and without the stopbank), although this difference was up to 0.22 m. The removal of the stopbank also allowed water to flow more swiftly back into the channel, as shown in Figure 8.16 after the 41st hour.

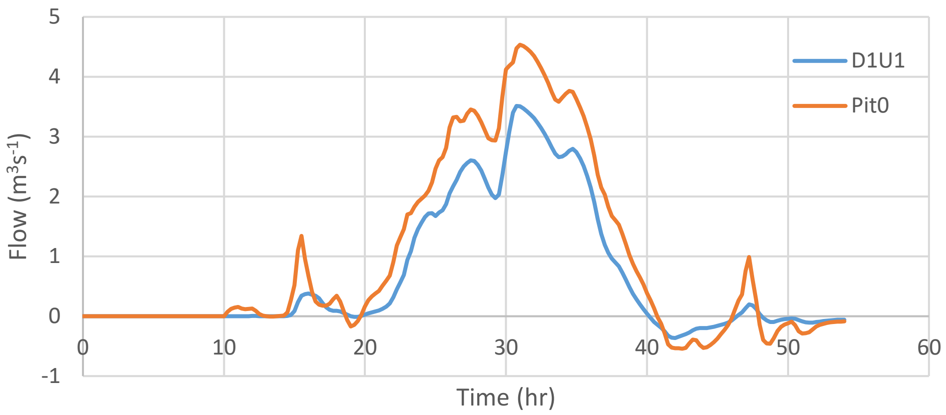


Figure 8.16. Flowrates in a 5-year event across the Pitfure stopbank

As the return period increased the difference in inundation extent between the D1U1 and Pit0 scenarios decreased. As with the MSG stopbank, this indicates that amount of protection the Pitfure stopbank provided diminished as the return period increased. The difference in flooding extent is shown in Figure 8.15. Given the maximum flooding extents were 7.6 km^2 and 17.8 km^2 for the 5 and 100-year events, the reductions in inundation areas represented decreases of 1.25% and 0.09% respectively.

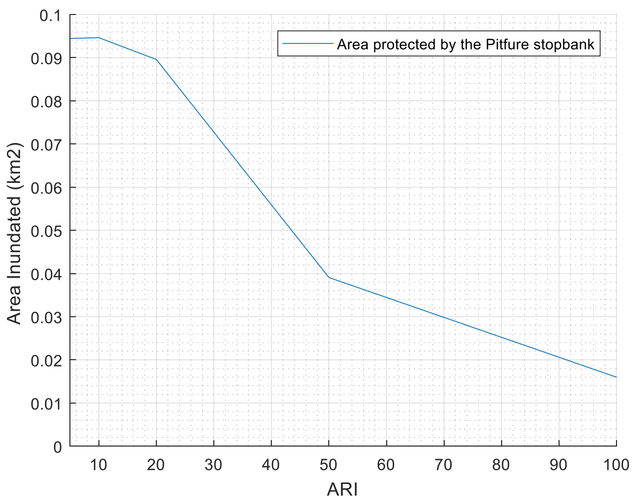


Figure 8.17. The difference in inundation extent caused by the Pitfure stopbank

The reason the Pitfure stopbank did not have a significant impact on the inundation extent was because of the overtopping in areas identified as having undulations during the condition assessment. This allowed flow into the adjacent land in a manner that was similar to when the stopbank was removed. Higher return period events also illustrated that the protective effect of Pitfure stopbank decreases as return periods became larger. The removal of the stopbank did not lessen the inundation downstream.

8.6. Confluence Stopbank Removed - Con0

The Confluence stopbank was also assessed as part of the study. The Confluence stopbank actually consisted of two stopbanks because the Wai-iti and Wairoa sides of the stopbank were not connected.

During the D1U1 scenario, the Wai-iti Confluence stopbank overtopped in two locations in a 5-year event. Flow from the Wai-iti River was also able to bypass the stopbank at the upstream end of the Wai-iti Confluence stopbank and inundate the adjacent land. The overtopping and flow around the end of the stopbank caused the confluence area to become significantly inundated. A notable exception to the flooding extent was a private residence on an elevated pad. The flooding extent in the different return periods is shown in Figure 8.18.

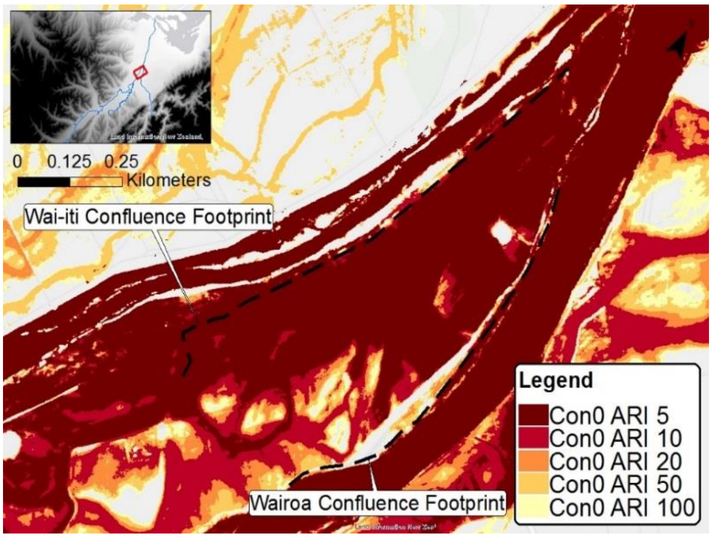


Figure 8.18. Overview of Con0 flooding extent

The impact of removing the Confluence stopbank could be assessed independently. In a 5-year event, when the Confluence stopbank was removed, the adjacent area was significantly inundated. The flow rate along the footprint of the Wai-iti side of the stopbank increased from a peak of approximately 30 m^3s^{-1} to approximately 50 m^3s^{-1} which is seen in Figure 8.19. Although this did cause the inundation extent to increase, it did not do so significantly. Much of the additional area inundated was the footprint of the stopbank. When the Confluence stopbank was removed, the average depth experienced within the adjacent area increased from 0.38 m to 0.47 m.

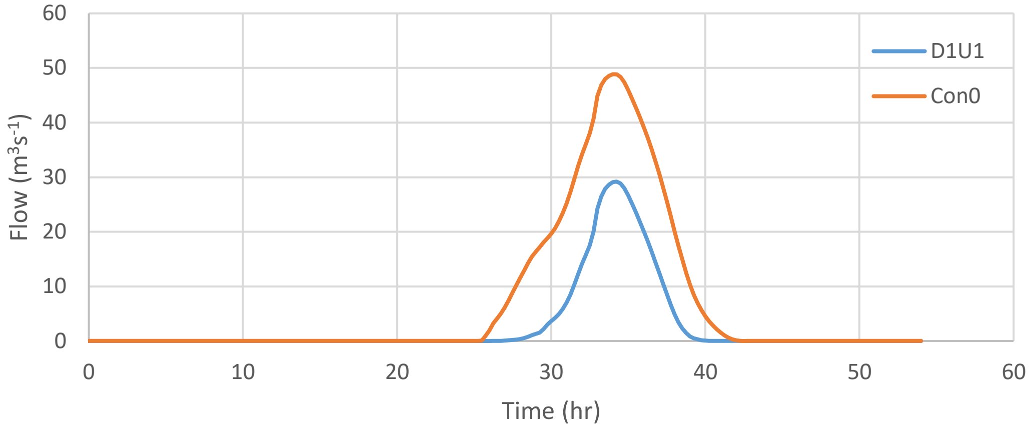


Figure 8.19. Flowrates in a 5-year event across the Confluence stopbank

The difference in inundation area decreased as the return period increased. The depths within the confluence also became larger, but with a decreased difference from 0.81 m and 0.83 m with and without the Confluence stopbank respectively. A comparison of the area inundated in a 5-year event is shown in Figure 8.20.

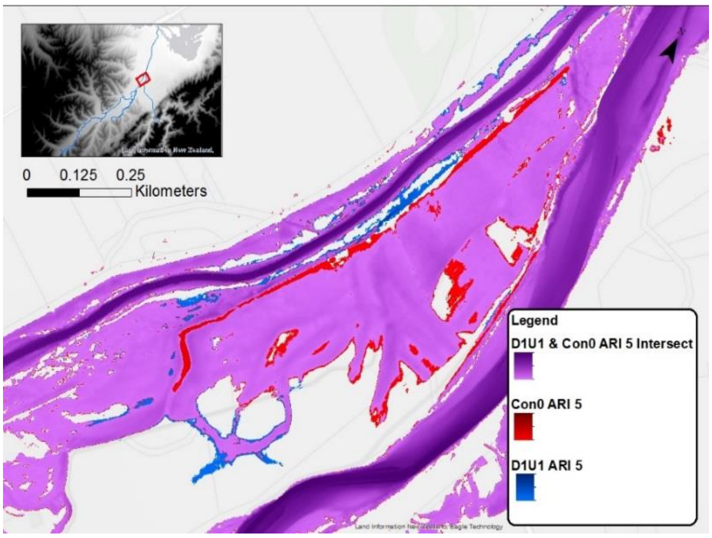


Figure 8.20. Flood extents of D1U1 and Con0 in a 5-year event at the Confluence stopbanks

At the upstream end of the stopbank there was another separate stopbank (240 m); however this stopbank was unconnected to the Confluence stopbank. Despite the second stopbank being higher, it was not effective in reducing the inundation extent as water was able to flow around it.

In summary, the removal of the Confluence stopbank did not increase the inundation area significantly. The presence of the stopbank, did reduce the depth in the adjacent area by 0.1 m in a 5-year event because less water was able to flow into the area. As the return period increased, the amount of protection provided by the stopbank reduced. In larger return periods the depths were similar to the D1U1 scenario.

8.7. Main Spring Grove Stopbank Maintained - MSGR

The maintaining scenario of the MSG stopbank scenario addressed the issues caused by the undulations in crest height. This aimed to fix the excessive overtopping by making the stopbank uniform as outlined in section 4.5. The flood extents for the different return periods are shown in Figure 8.21.

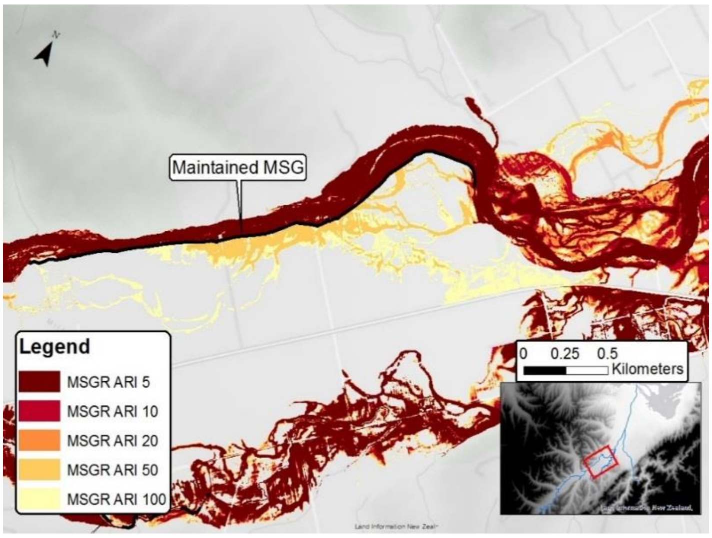


Figure 8.21.Overview of MSGR flooding extent

During a 5-year event, the maintained MSG stopbank prevented inundation in the adjacent land. At no point did the maintained MSG stopbank overtop or get bypassed. However, there was an increase in the inundation experienced at the airstrip. As the return period increased to a 20-year event, the stopbank was still able to prevent the adjacent land from becoming inundated. During a 50-year event, the maintained stopbank overtopped in the 30th hour of the design event, two hours before the flood peak. The overtopping occurred at one location, in the middle of the stopbank.

The overtopping occurred because greater flow was contained in the Wai-iti River. The MSG stopbank in previous scenarios overtopped which reduced the flow in the channel. In the MSGR scenario, because of the higher flow rate, the depth in the Wai-iti increased, which was able to overtop the maintained stopbank. The velocity was as high as 2 ms^{-1} at the point of overtopping. This velocity is likely to cause significant erosion (Hewlett et al., 1987) but this was not accounted for in the model. Figure 8.22 shows the flow in the Wai-iti channel.

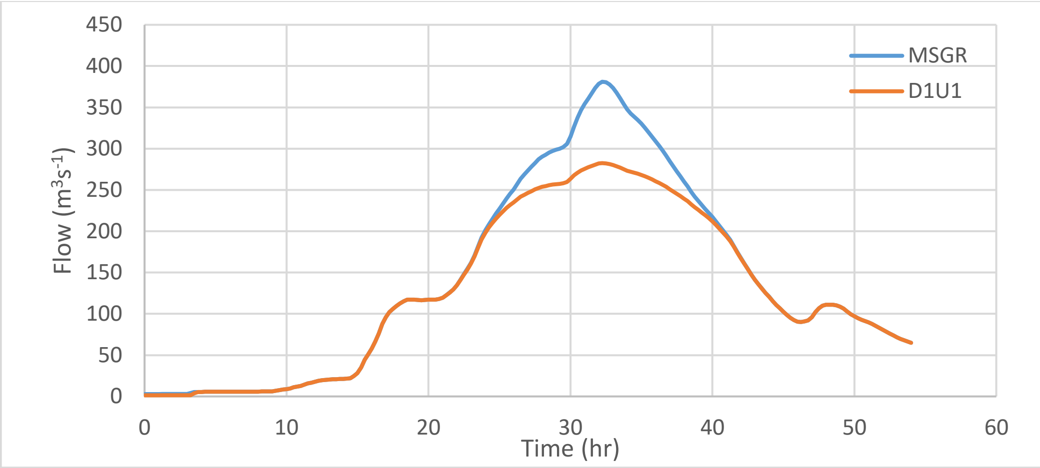


Figure 8.22. Flowrate in a 50-year event upstream of Wai-iti Bridge

During a 100-year event there was still a single point of overtopping. The flow overtopping the MSG stopbank was less than in the D1U1 scenario, causing less inundation of the adjacent land. Therefore, less flow was able to enter the Pitfure stream. This caused less inundation at the downstream end of the Pitfure Stream.

There was a minimal increase in the inundation extent at the airstrip in the 5-year event. However, in a 100-year event the inundation was more extensive because more water was contained in the channel by the maintained MSG stopbank, as shown in Figure 8.23. The water to the airstrip continued to flow across the adjacent land, causing more inundation west of the Council stopbanks.

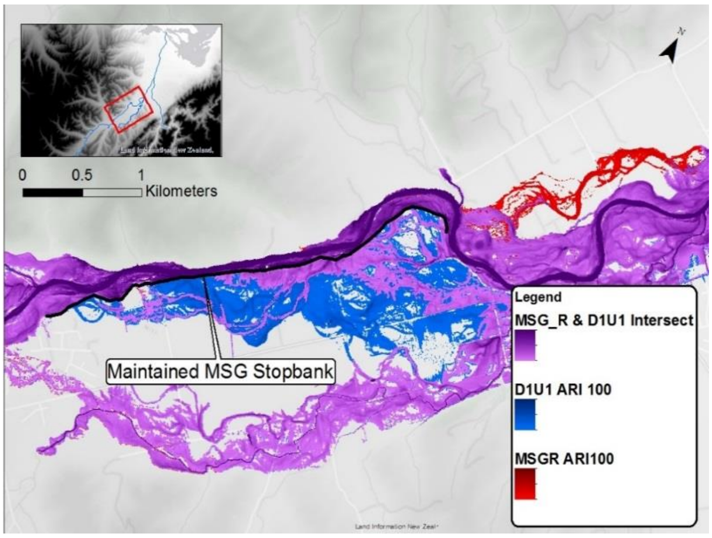


Figure 8.23. Flood extents of D1U1 and MSGR in a 100-year event at the MSG stopbank

The total area protected by maintaining the MSG stopbank in the different return periods is shown in Figure 8.24. The decrease in area protected occurred because of the overtopping, which led to inundation of the neighbouring production land.

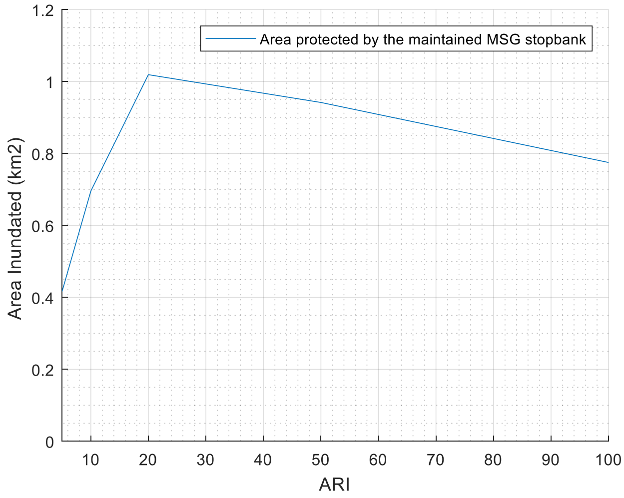


Figure 8.24. The difference in inundation extent caused by the maintained MSG stopbank

In summary, maintaining the MSG stopbank prevented land from being inundated. In 50-year events or greater, the maintained MSG stopbank overtopped in a single location. This scenario also showed that maintaining the MSG stopbank contained more flow in the Wai-iti channel, which resulted in more inundation at the downstream end of the stopbank.

8.8. Pitfure Stopbank Maintained - PitR

The major weakness of the Pitfure stopbank was the undulations along the crest. These were captured in the LiDAR and witnessed during the condition assessment. The undulations allowed water to pass unobstructed into the adjacent land. This scenario addressed this issue by replacing the current Pitfure stopbank with a structure with a uniform height of one metre. The stopbank was also extended such that it joined a natural terrace upstream. This scenario was used to investigate the benefits and consequences of maintaining the Pitfure stopbank.

During the 5-year event, the maintained Pitfure stopbank was able to prevent inundation without overtopping. The Brightwater side of the Pitfure remained completely free of inundation along the stopbank. The flooding extents for the different year event is shown in Figure 8.25.

Maintaining the Pitfure stopbank did cause a minor increase in flooding extent on the southern side channel because the maintained stopbank prevented flooding on the northern side. The southern side of the channel also experiences an increase in the average depth from 0.18 m to 0.20 m. The difference in flooding extent is shown in Figure 8.25.

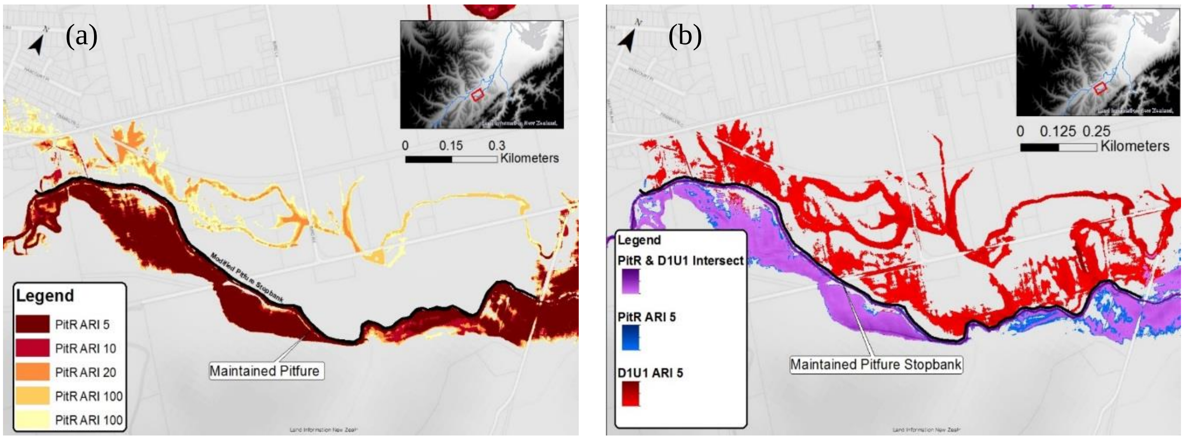


Figure 8.25. (a) Overview of PitR flooding extent (b) Flood extents of D1U1 and PitR in a 5-year event at the Pitfure stopbank

In a 10-year event, the maintained Pitfure stopbank overtopped at the connection where the upstream end of the stopbank joined the natural terrace. Here, there was a decrease in the crest height which allowed the overtopping. The overtopping had a low flow, however, it caused an area of approximately 0.018 km^2 to become inundated to a depth of mostly less than 0.1 m.

Increasing the return period further caused the area inundated by the connection to also increase. Despite this, the maintained Pitfure stopbank was able to contain the majority of the flow, which reduced the inundation area. A comparison of the flow across the footprint of the maintained stopbank is shown in Figure 8.26, demonstrating that the flooding caused by the connection was minor and that the maintained stopbank prevented a significant flow from entering the adjacent area. In all scenarios, in events with return periods greater than 20 years, some flow from the Wai-iti exited the channel and flowed through Wakefield before joining the Pitfure Stream. In the PitR scenario, this flow inundated a similar area to what was expected to if the Pitfure stopbank had overtopped.

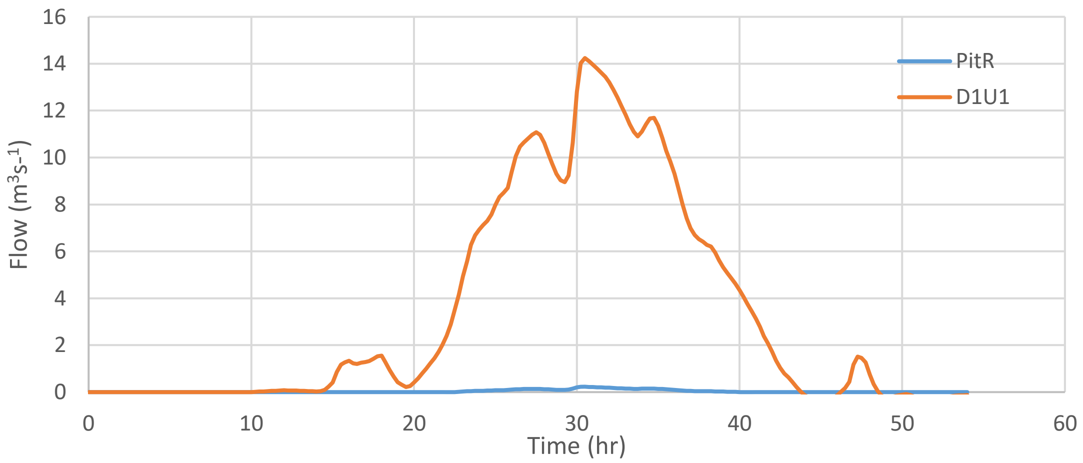


Figure 8.26. Flowrates in a 100-year event across the Pitfure stopbank

In a 100-year event, by protecting adjacent land on the north side of the stopbank, more water flowed into land south of the channel. This raised the average depth from 0.30 m in the D1U1 scenario to 0.40

m when the Pitfure was maintained. The total area protected by maintaining the Pitfure stopbank is shown in Figure 8.27.

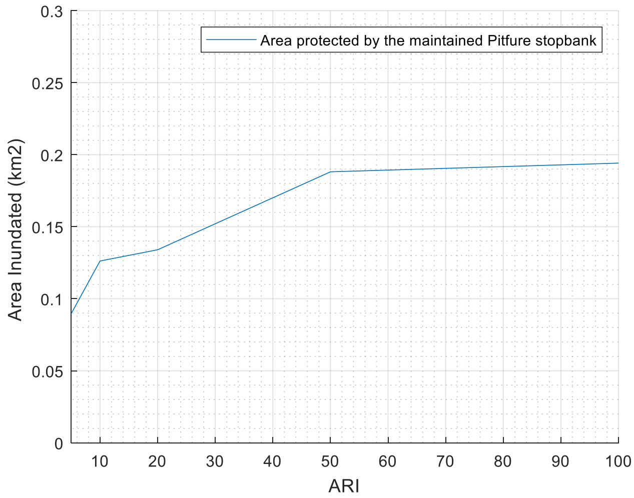


Figure 8.27. The difference in inundation extent caused by the maintained Pitfure stopbank

8.9. Inundation Summary

The Council stopbanks had the greatest impact on the flooding extent of the stopbanks. The Council stopbanks did not allow inundation of adjacent land and maintained approximately one metre of freeboard in a 100-year event. During larger events, some flow was able to bypass the stopbank by flowing around the upstream end of the Wai-iti side. The removal of the Council stopbanks, in a 100-year event led to 5.8 km^2 of additional area becoming inundated at an average depth of 0.50 m. The additional area was predominately located west of the stopbank, due to flow at the downstream end of the Wai-iti River.

The Confluence stopbank overtopped in two locations when it was present; however, the largest contributor to the inundation extent was the flow around the upstream end of the stopbank. This caused the area adjacent to the Confluence stopbank to become significantly inundated in a 5-year event. The removal of the Confluence stopbank did not cause significantly more area to become inundated as the confluence area was flooded by the flows bypassing the stopbank when it was present.

The MSG stopbank in its current state experienced overtopping in at least six locations, causing significant inundation of the adjacent land. At these points, the LiDAR captured undulations in the topography. These points were all located in areas of brambles or impenetrable cover identified in the condition assessment. When the stopbank was removed, there was an increase in flood extent (0.50 km^2 in a 5-year event). The area protected by the stopbank decreased as the return periods became larger. A scenario was created to investigate the effects of maintaining the stopbank. The maintained MSG stopbank was able to prevent inundation of adjacent land until the return period was increased to 50-years. This event caused one point of overtopping midway down the stopbank. Despite the overtopping, there was still a reduction in overall inundation extent (0.80 km^2 in a 100-year event). However, because more flow was contained in the Wai-iti River, there was an increase in inundation at the downstream end of the stopbank.

In its current state, the Pitfure Stopbank was unable to prevent inundation of the adjacent land due to undulations that caused the stopbank to overtop. When the stopbank was removed, there was not a significant increase in flooding extent. Because of this, a scenario was designed to analyse the benefits of maintaining the Pitfure stopbank as a uniform structure. This scenario showed a clear reduction in inundation extent of 0.2 km^2 in a 100-year event. In no events did the maintained Pitfure stopbank overtop however, due to the flow from the Wai-iti, a similar same region was inundated.

The differences in inundation area with respect to D1U1 are shown in Table 8.3

Table 8.3. Difference in maximum inundation areas with respect to D1U1 scenario

	Area Inundated with respect to D1U1 (km^2)							
ARI	D1U1	D0U0	D1U0	MSG0	Pit0	Con0	MSGR	PitR
5	7.51	+4.13	+0.46	+0.43	+0.03	+0.01	-0.49	-0.16
10	10.25	+4.99	+0.48	+0.44	+0.03	+0.00	-0.76	-0.20
20	12.61	+5.39	+0.26	+0.23	+0.02	+0.00	-1.09	-0.20
50	15.67	+5.66	+0.12	+0.06	-0.03	-0.04	-1.01	-0.26
100	17.83	+5.84	+0.09	+0.07	+0.02	+0.01	-0.77	-0.19

9. FLOOD IMPACT ASSESSMENT

The ability to assess the losses caused by natural hazards allows risk managers to make more informed decisions. As part of this study, a Riskscape impact assessment was undertaken. The Riskscape software is able to quantify the impact of natural hazards such as floods and was used to quantify the building impact of each flood event. The measures analysed for each scenario were damage state, exposed state, functional downtime, human displacement, and reinstatement cost. The methods for the Riskscape analysis are outlined in section 4.6.

9.1. All Stopbanks Present - D1U1

The D1U1 scenario acted as a baseline to which other scenarios could be compared. Exposed building for this analysis refers to any building in the assets layer that experienced any depth of flooding.

As the return period was increased from 5 to 100-years, the number of exposed buildings increased from 34 to 266. The average inundation depth for a 5-year event at the exposed buildings was 0.22 m. In a 100-year event the average depth was 0.31 m. Of the exposed buildings in the 5-year event, 14 were in areas identified as overpredicting inundation extent in the 1983 flood event.

The majority of these buildings were centred near the Brightwater and Wakefield townships. Adjacent to where the MSG stopbank overtopped, there were a number of buildings that were exposed to flooding. Figure 9.1 depicts the distribution of exposed buildings

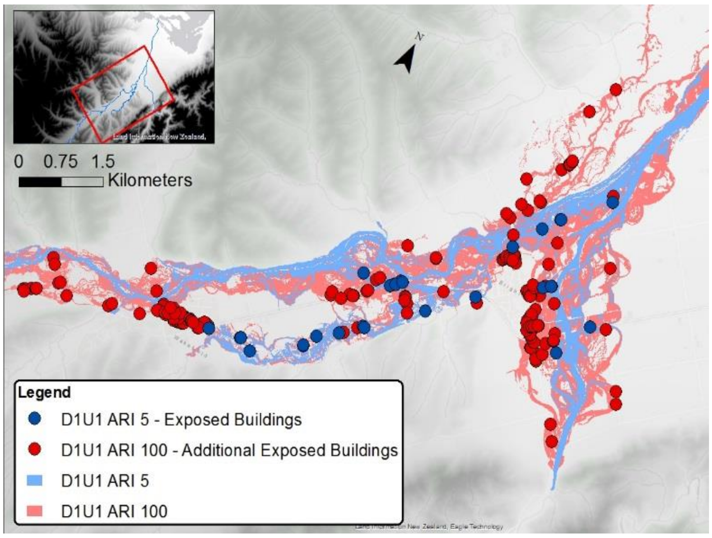


Figure 9.1. Distribution of exposed buildings in D1U1 scenario

Further analysis revealed the damage states of the exposed buildings. During the 5-year event, two building were moderately damaged. These were located near the Wai-iti/Wairoa confluence and downstream of the Pitfure stopbank. As the return period increased so did the severity of damage and the number of damaged buildings. No buildings were severely or critically damaged. In a 100-year event, of the thirty moderately damaged buildings; eighteen were due to flooding from the Wairoa River (mostly in Brightwater Township), ten were the result of flooding from the Wai-iti, and two were because of flooding from the Pitfure Stream. The damage states are shown in Figure 9.2.

Similar to the damage state, as the return period increased so did the severity and amount of human displacement. In higher return periods the properties had increased damage with longer reoccupation times. Because of this, they were moved into the higher brackets (more than 6 months until possible reoccupation), hence the decrease in buildings with reoccupation times of ‘up to six months’. The amount of human displacement expected for the different return periods is shown in Figure 9.2.

Further analysis revealed the reinstallment costs also increased as the return period increased (Figure 9.2). For 20-year events and greater, the damages were mostly between $200,000 and $500,000. For the 100-year event, there were two buildings that suffered between $500,000 and one million dollars of damage. These buildings were Nelson Sea-Doo & Can-Am (a Canadian vehicle dealership) and a residential property.

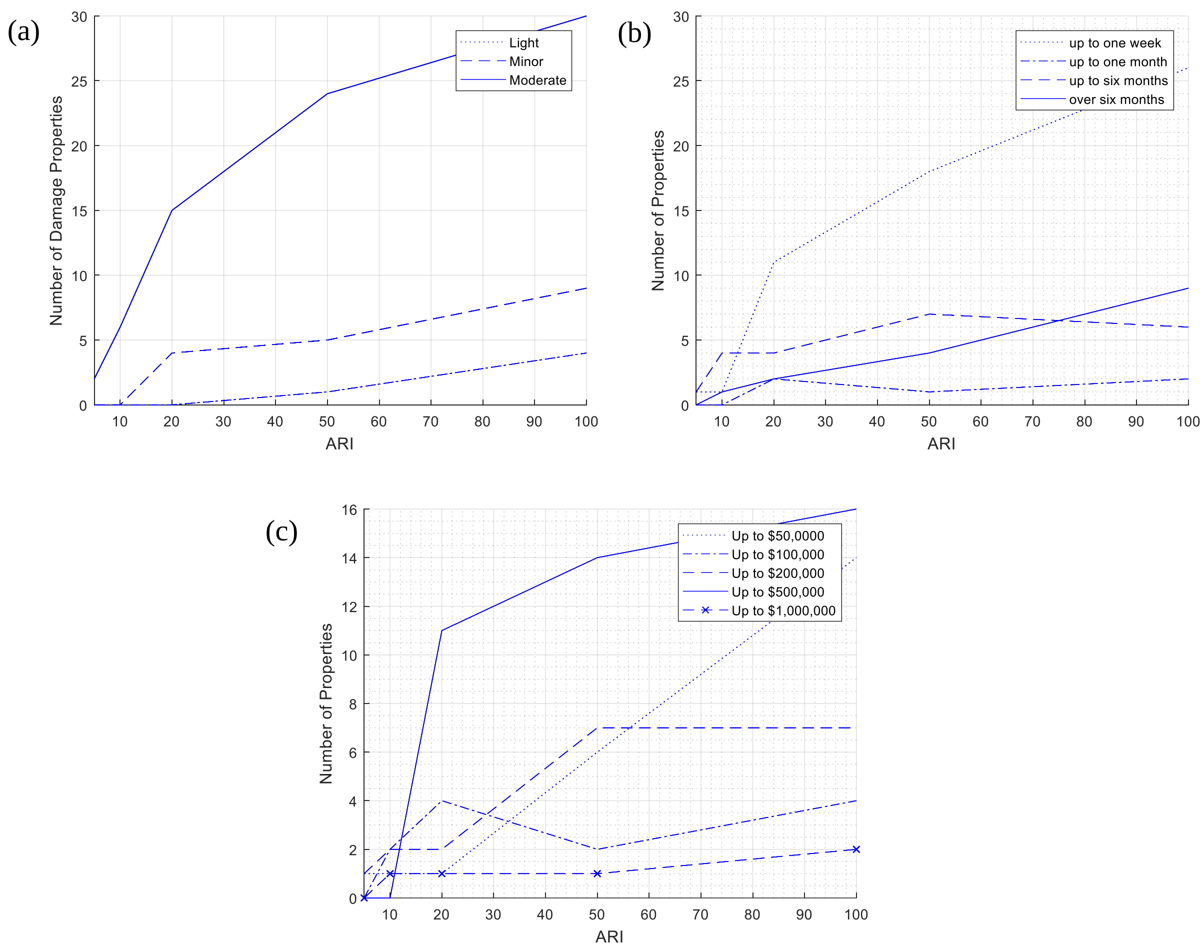


Figure 9.2. D1U1 scenario: (a) Building damage states (b) building reoccupation time brackets (c) building reinstallment cost bracket

The overall reinstallment cost was divided into different categories according to Riskscape. Table 9.1 shows that with the current terrain, a 100-year event would cause approximately $8.7M worth of damage. In the smaller events, the majority of the building reinstallment costs were due to the asset repair costs of the two buildings moderately damaged. In events larger than 20 years, stock replacement costs contributed the largest share of damage costs (>40%). Nelson Sea-Doo & Can-Am alone had $600,000 worth of stock replacement costs. The bulk of the stock costs came from twelve closely clustered buildings located around Fonterra Brightwater on Factory Road. These buildings all suffered $225,000 worth of stock loss. This damage was because the Wairoa River was able to inundate the Fonterra building complex in events larger than 20 years.

Table 9.1. Distribution of building reinstallment costs by category

	ARI 5	ARI 10	ARI 20	ARI 50	ARI 100
Total Damages (NZD Thousands)	159,000	1,075,000	4,576,000	6,352,000	8,714,000
Stock Replacement (%)	0.0	0.0	49.9	44.1	40.1
Asset Repair (%)	56.0	53.6	25.2	28.0	30.3
Contents Repair (%)	20.9	35.2	11.4	12.9	13.3
Disruption Cost (%)	13.9	6.2	8.4	9.3	10.1
Clean up Cost (%)	8.6	4.7	3.8	4.3	4.3
Plant (%)	0.6	0.1	1.2	1.1	1.1
Vehicle Cost (%)	0.0	0.2	0.1	0.3	0.9
Services (%)	0.0	0.0	0.0	0.0	0.0

In summary, as the flood events became progressively larger, the amount of area inundated increased. This flooded more houses, caused more structural damage, longer times until reoccupation, and greater reinstallment costs. There was an increase in costs between a 10 and 20-year event as a result of water exiting the Wairoa channel and inundating twelve buildings at Fonterra Brightwater.

9.2. No Stopbanks Present - D0U0

This scenario represented the worst-case situation where all stopbanks were removed. As a greater area was inundated, the number of exposed buildings increased. The D0U0 scenario had on average 48 more properties exposed regardless of the return period, shown in Figure 9.3. The additional properties exposed, with one exception, were located adjacent to the Council stopbanks.

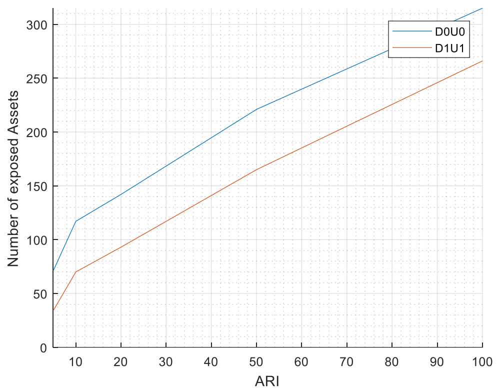


Figure 9.3. Number of exposed building in D1U1 & D0U0 scenarios

Of the 315 buildings exposed to the flooding in a 100-year event, 60 suffered damages. During the 5-year event, there were three more buildings with moderate damage each and two more houses with minor damage. During a 100-year event, twelve additional buildings were moderately damaged. These were predominantly located west of the Council stopbanks and damaged as a result of the west Council stopbank being removed.

As in the D1U1 scenario, there were no instances of severe or critical damage. However, the number of light, minor, and moderately damaged properties was higher, as illustrated in Figure 9.4. The additional damaged properties, as with the 5-year event, were located in areas that were previously protected by the Council stopbanks.

The same properties (Nelson Sea-Doo & Can-Am and a residential property) had damages between $500,000 and $1,000,000. Additionally, there were three more properties damaged up to $500,000. The greatest increase was in houses with up to $50,000 of damages. As explained earlier, the decline in one bracket indicates an increase in an upper bracket, shown in Figure 9.4. The decrease in inundation at the airstrip resulted in an approximate saving of $6000 in a 100-year event at the one building affected.

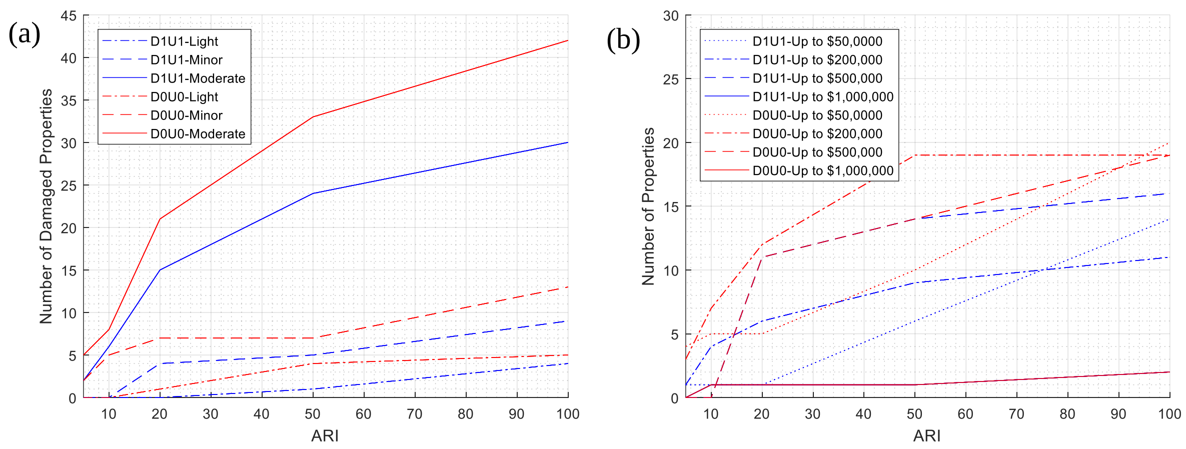


Figure 9.4. D0U0 & D1U1 scenarios: (a) building damage states (b) building reinstallment cost

Because the number of buildings damaged increased, the total cost of the flood events increased. Removing all the stopbanks increased the cost of a 100-year event by $1.6M. The difference in cost was in damaged buildings adjacent to the Council stopbanks. The five buildings that experienced the greatest increase in reinstallment cost were all west of the Council stopbanks and made up 62% of this difference.

This scenario showed that the stopbanks prevented, on average, approximately 50 buildings from being exposed to flooding. In a 100-year event, the stopbanks protected an additional 12 buildings from moderate damage (30 buildings in D1U1) and five more from minor and light damage. The additional damaged buildings were generally on the western side of the Council stopbanks. The result of this was a reduction in the reinstallment cost of $1.M, a reduction of 15% compared to the D0U0 scenario ($10.3M in D0U0).

9.3. No Undocumented Stopbanks Present - D1U0

As with the inundation extent, comparing this scenario to the D1U1 case allowed the impact of all the undocumented stopbanks to be assessed. By comparing this scenario to the D0U0 scenario, the impact of the Council stopbanks was also assessed.

Compared to the D1U1, there was an average increase of two buildings. Therefore, the removal of the undocumented stopbank did cause a minor increase in exposed building. In comparison to the D0U0 scenario, there was an average reduction of 47 exposed buildings.

For events smaller than 50 years, the damage states of the D1U0 scenario were identical to the D1U1 scenario. This illustrated that for smaller events, collectively the undocumented stopbanks did not have a significant impact on building damage. In a 100-year event, the removal of the undocumented stopbanks caused a building at the confluence to become severely damaged. This building was not severely damaged in the D0U0 scenario. This may have been caused by the Council stopbanks containing the flow within the Wai-iti and the removal of the Confluence stopbank allowing the additional flow to spill into the confluence area. Additionally, in a 100-year event, there was one extra lightly damaged building near where the MSG stopbank overtopped.

Compared to D0U0, the presence of the Council stopbanks (on average) reduced the number of moderately damaged buildings by 28%. In a 5-year event, the number of moderately damaged buildings was reduced from five to two, and prevented any buildings from becoming lightly damaged. The majority of the additional building damage was along the west Council stopbanks along Waimea W Road. The distribution of moderately damaged building is shown in Figure 9.5. The human displacement in this scenario was larger than the D1U1 scenario but less than the D0U0 scenario, which followed the same trend as the damage states.

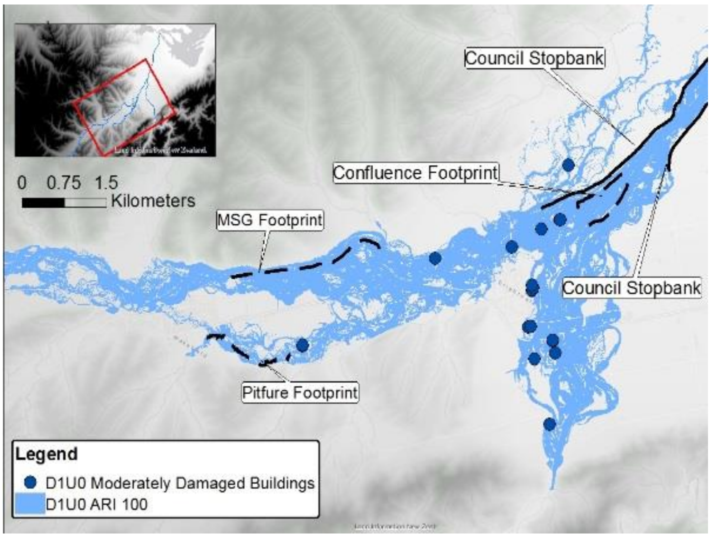


Figure 9.5. Distribution of moderately damaged buildings in D1U0 scenario

Compared against the D0U0 scenario, the total reinstallment costs incurred by the absence of the Council stopbanks increased as the return period increased. In 5 and 100-year events, the prevented building damage was $335,000 and $1.6M. This represented savings of 65% and 15% for their respective events. The $1.6M in additional costs were attributed to the properties west of the Council stopbanks being damaged. The majority of these buildings were damaged and had reinstallment costs less than $100,000. In comparison to the D1U1 case, the undocumented stopbanks prevented approximately $30,000 worth of damages on average. The difference in cost was spread throughout the domain.

This scenario showed that the undocumented stopbanks did not protect many buildings and therefore did not reduce the overall damage states or reinstallment costs of flood events. This was because the undocumented stopbanks primarily protected land that had a low density of buildings. The inundation extent showed the current undocumented stopbanks were ineffective at reducing flood extent. These factors resulted in the few properties near the undocumented stopbanks being affected by flooding when the stopbanks were present. When the stopbanks were removed the damage did not increase significantly. The Council stopbanks, in contrast, protected twelve buildings from moderate damage. Their removal incurred approximately $1.6M worth building reinstallment costs.

9.4. Main Spring Grove Stopbank Removed - MSG0

This scenario assessed the individual impact of the MSG stopbank to buildings. In all return periods, the number of exposed houses was similar to the D1U1 scenario. In a 10-year event, six buildings were additionally exposed to the flooding along SH6 because the MSG stopbank was removed. However, the removal of the MSG stopbank also prevented three buildings at the airstrip from becoming exposed to flooding, as the flow was reduced.

Translating the exposed buildings to damage states, it was seen that the number of buildings exposed was also similar to the D1U1 scenario. The moderately damaged buildings in all return periods were predicted to be the same. The damage states are shown in Figure 9.6.

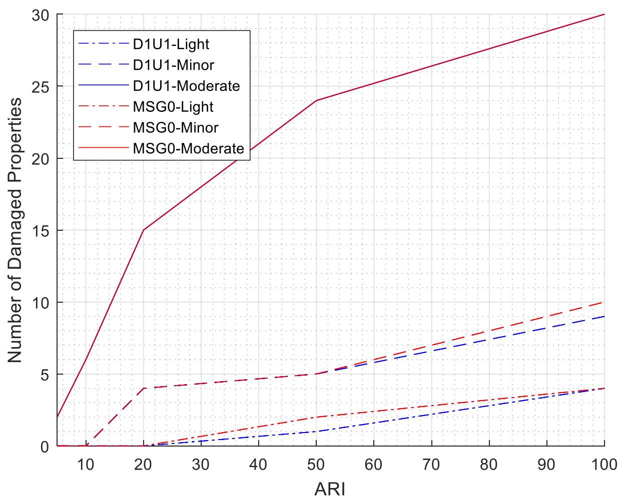


Figure 9.6. Building damage states in MSG0 & D1U1 scenarios

The human displacement values were similar to those of the D1U1 scenario. Although in a 20-year event (where the event was large enough to cause significant inundation but small enough for the MSG stopbank to still be somewhat effective) the removal of the MSG stopbank caused one building to transition from a reoccupation time of 'up to a month' to 'up to six months', illustrated in Figure 9.7.

The effect that the MSG stopbank had on building reinstallment cost was minimal. The difference caused by removing the MSG stopbank in total damages, in a 100-year event, was $26,300. This corresponded to savings of 0.3%. Other return periods yielded similar results of less than 1% increased damage by removing the MSG stopbank.

The lack of difference in total reinstallment cost was for two main reasons. Firstly, the undulations in the MSG crest caused buildings to be damaged in the D1U1 scenario. Therefore, when the stopbank was removed there was not predicted to be a significant increase in damaged buildings. Secondly, the MSG stopbank primarily protected production land with a low density of buildings, lowering the potential for difference as there were fewer buildings to be damaged in the affected area.

Further analysis showed that the removal of the MSG stopbank caused a slight increase in the number of properties in higher reinstallment cost brackets. However, the majority remained the same as the D1U1 case, shown in Figure 9.7.

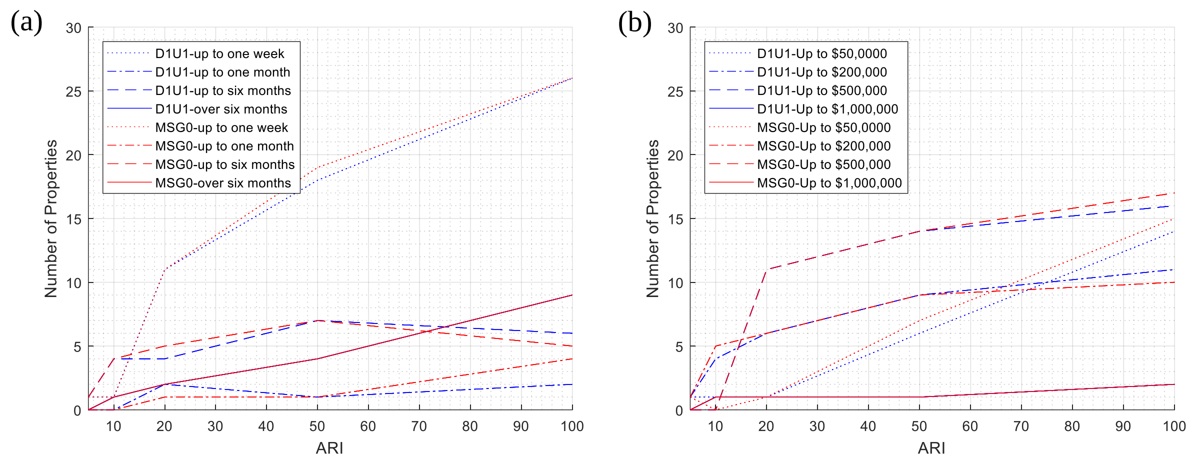


Figure 9.7. MSG0 & D1U1 scenarios:(a) building reoccupation time (b) building reinstallment cost

In summary, the removal of the MSG stopbank did not have a significant effect on the predicted damages incurred from the flooding. This was because the stopbank has a number of depressions that allowed water through the stopbank. Although the removal of the stopbank increased the inundation extent, this did not significantly increase the building damages.

9.5. Pitfure Stopbank Removed - Pit0

This scenario assessed the individual impact of the Pitfure stopbank to buildings. The Pitfure stopbank was previously shown to slightly reduce the flooding extent. When the stopbank was removed, the losses were similar to the D1U1 scenario. This was because there were few buildings near the Pitfure stopbank and they were set a respectable distance from the Pitfure Stream. These factors, coupled with the lack of influence the Pitfure stopbank has on flooding extent, resulted in predicted building damage states that were the same as the D1U1 scenario, shown in Figure 9.8.

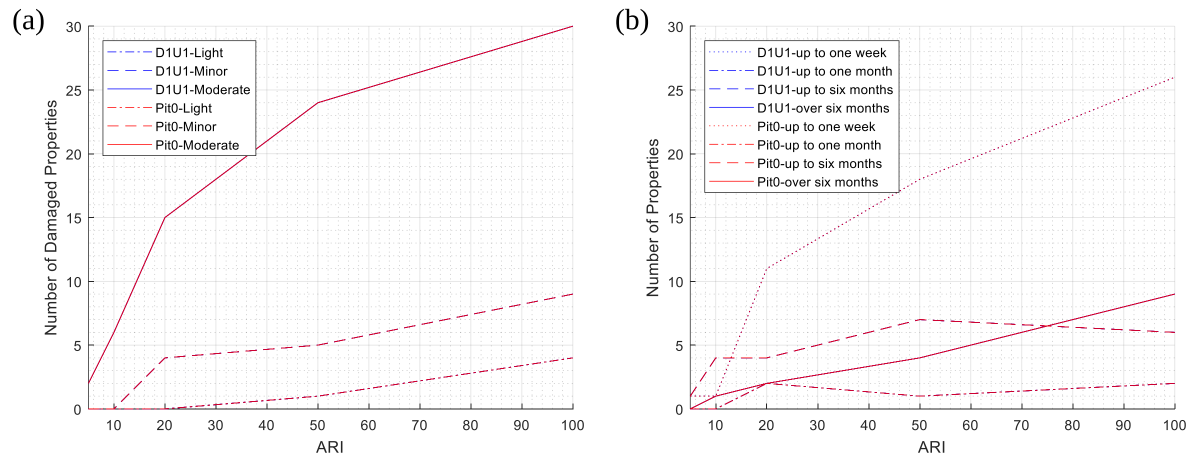


Figure 9.8. Pit0 & D1U1 scenarios: (a) building damage states (b) building reoccupation time

The human displacement caused by the removal of the Pitfure stopbank was also predicted to be the same as the D1U1 scenario, regardless of the return period, Figure 9.8. The affected properties were primarily located in the Brightwater Township and affected because of flooding from the Wairoa River.

There were also a number of houses displaced around Spring Grove from water overtopping the MSG stopbank. The spread of human displacement resulting from a 100-year event is seen in Figure 9.9.

For numerous reasons, the majority of the human displacement was a consequence of the Wairoa flooding rather than the Wai-iti and Pitfure. The Wairoa was a much larger river than the Wai-iti (flows in a 100-year event were 1474 and 512 m^3s^{-1} respectively), and this meant that the Wairoa had greater potential to cause damage. The Wairoa was also not actively protected by formal stopbanks at its upper reaches, relying instead on natural terracing. There was also a greater density of buildings near the Wairoa flooding extent in Brightwater Township than near the Pitfure. The building density around these areas was also much sparser as the land was primarily utilised for production purposes, lending itself to large paddocks with few buildings. Because of these factors, when the Pitfure Stream was removed the inundation did not cause as many losses. The distribution of human displacement in a 100-year event is shown in Figure 9.9.

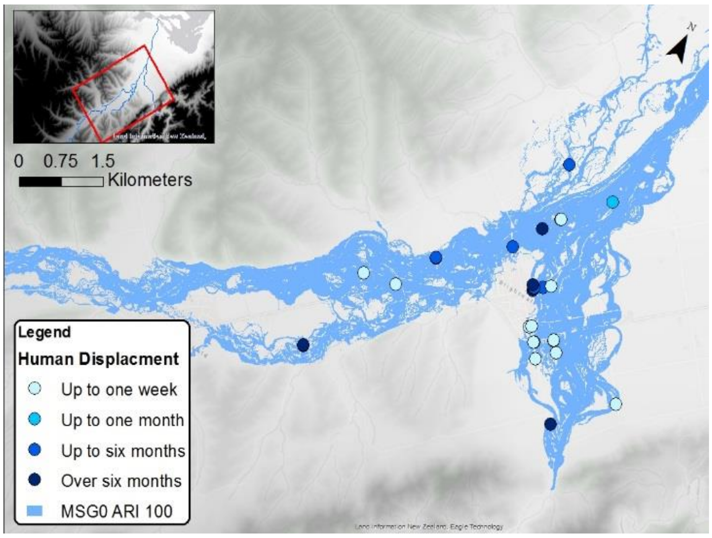


Figure 9.9. Distribution of building human displacement in MSG0 scenario

The removal of the Pitfure stopbank did not increase the reinstallment costs for the D1U1 scenario. The cost incurred was estimated at $159,000 for a 5-year event and $8,714,000 for a 100-year event. A breakdown of the total costs is shown in Table 9.2 for a 100-year event. The largest bracket of damage was from the 16 properties with up to $500,000 worth of damages (46% of total reinstallment cost). Twelve of these buildings were closely clustered by Riskscape at Fonterra Brightwater. Two-thirds of the damage to these buildings was stock damage. In reality, these buildings were not as closely clumped as they were in Riskscape, but there were a number of facilities within the clusters vicinity.

Table 9.2. Building Reinstallment cost brackets

Reinstallment Bracket	Total cost	Number of Buildings
Up to $50,000	$377,000	14
Up to $200,000	$1,397,000	11
Up to $500,000	$5,241,000	16
Up to $1,000,000	$1,699,000	2
Over $1,000,000	$0	0
Total	$8,713,000	43

The lack of building impact was, in part, due to the depressions seen in the stopbank through LiDAR and during the condition assessment. These allowed water to pass through the stopbank essentially unimpeded. Therefore, the removal of the stopbank did not cause significantly more area to become inundated. As a result, the damage states, human displacement, and losses of buildings did not decrease when the stopbank was present. Hence, in its current state, the Pitfure stopbank is not predicted to significantly reduce the impact to buildings.

9.6. Confluence Stopbank Removed - Con0

This scenario assessed the individual impact of the Confluence stopbank to buildings. The flood extent analysis found that the Confluence stopbank did not have a significant impact on inundation extent. Because of this the building impact results were also similar to the D1U1 scenario.

A notable difference from the other scenarios was that, in the 100-year event, the removal of the Confluence stopbank caused severe (irreparable structural) damage to one building. The damaged building was a residential building in the centre of the confluence area. The increased damaged is most likely due to the inundation duration being two hours longer with the stopbank removed (15 hours total). The differences in the other measure were not significantly different from the D1U1 scenario. The depth was the same and the velocity experienced at the point increased slightly (0.47 ms^{-1} and 0.56 ms^{-1} with and without the Confluence stopbank respectively). The difference in reinstallment cost for this building was $2,500 ($68,300 and $70,800 with and without the stopbank respectively). It should be noted that where the building was present in Riskscape, there was none in reality. However, a building was located nearby of a raised platform that was not exposed to inundation.

There were several more buildings that transitioned from insignificantly damaged to lightly damaged in the event of the Confluence stopbank being removed. These buildings were mostly located in Brightwater and further investigation identified that the maximum depths, velocities and durations were identical in the D1U1 and Con0 scenarios. These were all the inputs for the Riskscape software, meaning the increased damage was unexplained.

In summary, the removal of the Confluence stopbank did not increase the area inundated or the depths however, the presence of the stopbank did slow the inundation. This action prevented a building from becoming severely damaged. The additional costs incurred from removing the Confluence stopbank were estimated at $2,500.

9.7. Main Spring Grove Stopbank Maintained - MSGR

As previously documented, the MSGR scenario reduced the flood extent by up 1.0 km^2. This analysis addressed the impacts of this on buildings. The difference in exposed buildings between D1U1 and MSGR was between five and eighteen. In a 5-year event, the non-exposed buildings were at the intersection of Telenius and SH6. The distribution of the non-exposed buildings is shown in Figure 9.10.

As the return interval increased, there were fewer exposed buildings in Brightwater. This was because the maintained stopbank stopped the flow from the Wai-iti to the Pitfure, and therefore there was less flow in the Pitfure Stream. This led to less flooding near the Pitfure and thus less exposure in Brightwater Township. As a consequence of the increased flow contained in the Wai-iti in the larger events, three extra buildings were exposed near the airstrip. The number of exposed building is shown in Figure 9.11.

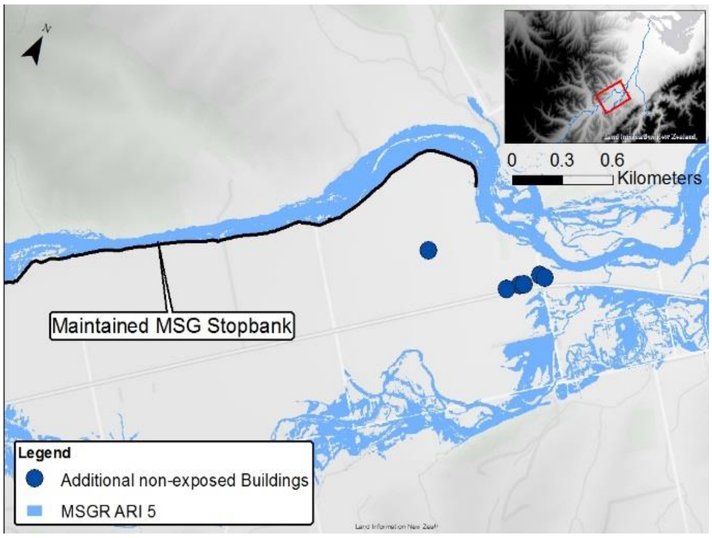


Figure 9.10. Distribution of buildings not exposed to flooding by maintaining the MSG stopbank

Despite the reduction in the inundation area and exposed buildings, generally, the D1U1 and MSGR scenarios had the similar damage states for each return period, however, there were two exceptions. Firstly, in 10 and 50-year events, there was an increase in moderately damaged buildings. During the 50 year event, the additional flow contained by the maintained stopbank caused a building at the airstrip to become moderately damaged. In the D1U1 scenario, it was lightly damaged. Secondly, during a 10-year event near the confluence area, there was an additional moderately damaged building. Similarly, because of the maintained MSG stopbank, more flow was contained in the Wai-iti River, which led to the building being inundated 15 minutes earlier and becoming moderately damaged. Other than these two differences, the moderate damage states of the D1U1 and MSGR scenarios were estimated to be the same, illustrated in Figure 9.11.

Because of these two differences, maintaining the MSG stopbank was estimated to increase the overall building damage states. The result of the increased damage states caused an increase in the amount of human displacement experienced. This increase was mostly due to the transitions between the lower classifications of ‘up to a week’ and ‘up to a month’.

The building reinstallment cost was not significantly different from the D1U1 scenario, shown in Figure 9.11. The areas protected by the maintained stopbanks were primarily production land with a few buildings interspersed. In a 10-year event, maintaining the MSG stopbank caused an increase in building reinstallment costs. The majority of the $50,000 difference was because of the additional moderately damaged building at the confluence area. Other than this exception, the difference in reinstallment costs was insignificant.

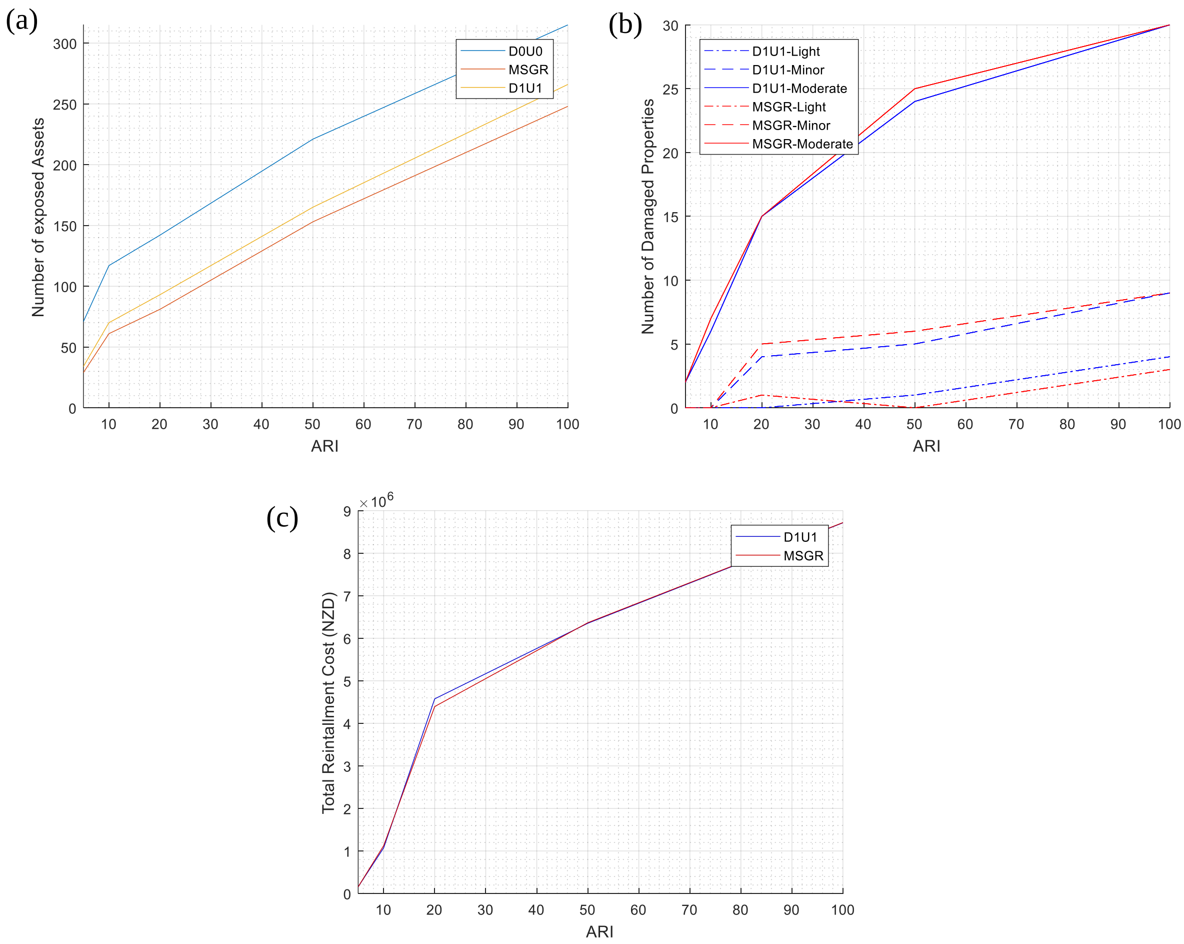


Figure 9.11. MSGR & D1U1 scenarios: (a) number of exposed buildings (b) building damage states (c) total reinstallment cost

In summary, maintaining the MSG stopbank did prevent land from being inundated. Overall, this prevented several buildings near Telenius Road becoming exposed, however, the damage states were estimated to not differ from the D1U1 scenario. Two extra buildings were moderately damaged in the

10 and 50-year events. These buildings were at the airstrip and the confluence. Because there were few differences in the affected buildings, the overall reinstallment costs were nearly identical to the D1U1 scenario. Because the impact analysis was restricted to building damage and did not incorporate the impact to production land, vineyards, etc. it was difficult to determine with certainty if maintaining the MSG stopbank reduced the overall losses experienced.

9.8. Pitfure Stopbank Maintained - PitR

This analysis addresses the impacts of maintaining the Pitfure stopbank to building damage and reinstallment cost. The maintaining of the Pitfure stopbank resulted in an average drop of eleven exposed houses for each return period. The non-exposed buildings were located mostly along Higgins Road. The spread of the exposed building is shown in Figure 9.12.

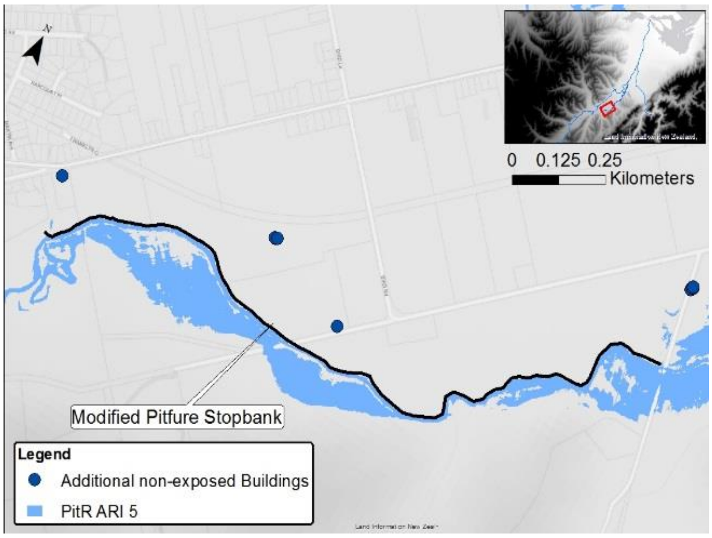


Figure 9.12. Distribution of buildings not exposed to flooding by maintaining the Pitfure stopbank

The only difference between the PitR and D1U1 scenarios in terms of damage states was that maintenance of the Pitfure stopbank prevented one building from moderate damage. This building was located near the downstream end of the Pitfure stopbank in a Riskscape cluster of four buildings. The building experienced 0.35 m and 21:30 hours less of flooding.

The reason that there was not a more discernible difference in the number of damaged buildings was that the area protected from flooding was predominantly production land, containing approximately 20 buildings, only two of which were damaged during the D1U1 scenario. Because of this, it was difficult to further reduce building losses.

In the D1U1 scenario, the building was unable to be reoccupied between one and six months for a 5 and 10-year event. When the return period was greater (20, 50, 100-year events), this increased to over six months. However, when the Pitfure stopbank was maintained, the displacement period was reduced for smaller events (5, 10-year event) to between a week and month. For the larger events, it was reduced to between a month and six months. As with the damage states, other than this building, the human displacement caused by the flood was the same as the current scenario.

In contrast to the MSGR scenario, the total reinstallment cost decreased due to the Pitfure stopbank being maintained, as illustrated in Figure 9.13. The savings on average were approximately $150,000. The savings were the result of two buildings suffering fewer losses. These buildings were located in the cluster at the downstream end of the Pitfure stopbank.

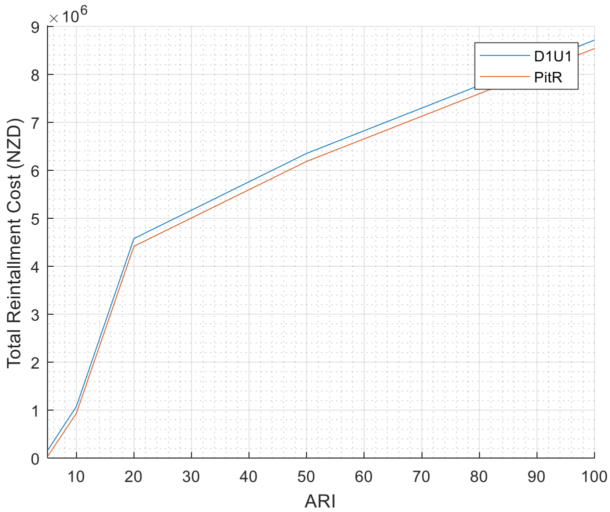


Figure 9.13. Building reinstallment cost for PitR and D1U1 scenarios

For a 100-year event, one of the buildings suffered losses of $190,000 and $75,000 in the D1U1 and PitR scenarios respectively. The increase came from an increase of $50,000 in both asset and content damage.

A second building was moderately damaged in the D1U1 scenario; however with the maintenance of the Pitfure stopbank it was estimated to be not damaged. Located at a point 4 m away from the previous building, a building suffered $60,000 worth of costs. Most of this loss was asset damage. Other than these two points, the difference in reinstallment costs for the buildings was insignificant.

The result of the Pitfure being maintained was that one less building, near the stopbank at Bridge Valley Road, was moderately damaged in all simulated return periods. Other damage states were identical to the current scenario. Because one less building was moderately damaged, the human displacement for the building was decreased. The maintenance of the Pitfure stopbank also decreased the reinstallment cost by $150,000. This arose from two buildings; one was the moderated damaged building that suffered no damage in the maintained scenario, and the other was in the same Riskscape cluster. The savings were predominantly from less damage to building assets. Other than these two buildings, there were no significant savings to buildings from maintaining the Pitfure stopbank. This was because the land protected was production land with only 21 buildings on it. Of these 21 buildings, two were affected by flooding in the D1U1 scenario. This made it difficult to significantly reduce the effect of flooding on building because the main damage centre was the Brightwater Township.

9.9. Riskscape Impact Summary

Various stopbank scenarios were investigated to assess the effects of removing and maintaining the stopbanks on flooding extent and the implications of this on the building assets within the region.

The undocumented stopbanks were estimated to not have a significant impact on reducing the overall damage states of buildings. This was because the area protected by the undocumented stopbanks was primarily production land with a low density of buildings. Because the undocumented stopbanks were estimated to have little impact on the overall damage states, the approximate reinstallment costs did not significantly increase when the stopbanks were removed. The Council stopbanks, in contrast, were estimated to prevent approximately an additional $1.6M damages to buildings (total cost in the D1U1 scenario $8.7M) and approximately reduced the number of moderately damaged buildings by 28%.

The scenarios also simulated the impact on buildings of maintaining of the MSG and Pitfure stopbanks. Maintaining the stopbanks did prevent inundation, however, because the land was predominately used for agricultural/horticultural purposes, the effect of maintaining the stopbanks had on damage and reinstallment costs to buildings was minimal for both scenarios.

These modelling results indicated that the net effect of removing or maintaining the undocumented stopbanks was minimal compared to the effect of removing the Council stopbanks. During this analysis, the losses experienced by production land were not considered as the losses depended on a number of factors which were difficult to quantify such as season, crop type, and timing of the flood. A summary of the building reinstallment costs is presented in Table 9.3

Table 9.3. Difference in building reinstallment cost with respect to D1U1 scenario

	Total reinstallment cost with respect to D1U1 (NZD Thousands)							
ARI	D1U1	D0U0	D1U0	MSG0	Pit0	Con0	MSGR	PitR
5	159	+377	+30	+1.1	+17.9	+9.3	-1.5	-124
10	1070	+507	+22	+2.4	+14.4	+23.3	+52	-142
20	4570	+719	+35	+21.2	+9.3	-7.6	-181	-160
50	6350	+1290	+22	+9	+13.7	+26.6	+13.4	-167
100	8710	+1600	+16	+26.3	-0.2	+2.5	+5.3	-176

9.10. Estimation of Stopbank Potential Impact Classification

It should be noted that the NZSOLD method for determining a Potential Impact Classification (PIC) specifically excludes stopbanks. It should also be noted that a complete PIC assessment was not completed as it would require knowledge of breach mechanisms that were beyond the scope of this thesis. Because of these limitations, the following PIC assessment should be considered complementary to the Riskscape impact assessment. The method and related tables are found in section 4.7.

The assessed stopbanks ideally would be considered when they were retaining the largest volume of water and therefore, when they have the largest impact. The undocumented stopbanks overtopped in all the modelled return periods they were considered at the smallest return period (5-years). In none of the modelled scenarios did the Council stopbanks overtop. Because of this, they were assessed in the largest simulated return period which was a 100-year return period. A consequence of this was an unconservative assessment of the Councils stopbank.

Instead of considering a breach and the associated mechanisms (which are required if the method were to be applied in the way it was intended) the inundation extents from removal scenarios were considered instead.

9.10.1. Council Stopbanks Assessed Damage Level

The removal of the Council stopbanks within the model led to an additional eight residential properties becoming moderately damaged. These buildings were all located west of the Waimea and Wai-iti Rivers. Four of these buildings were situated approximately 450 m from the river channel near the downstream edge of the domain at the intersection of River Road and Waimea W Road. Because of the number of moderately damaged buildings, the stopbanks had a Major impact on residential buildings.

Although a complete investigation into the impact on critical infrastructure was not conducted, an analysis of the water and wastewater features found that the bores along the bottom section of the stopbank were unlikely to experience flooding due to the removal of the Council stopbanks. This was because the bores were located along the upstream side of the stopbank and will likely experience flooding if the stopbanks are present. From this analysis, the damage to critical infrastructure was determined to be Minor.

An additional 5.6 km^2 of land was expected to be inundated as a result of the council stopbanks being removed. The primary land use affected was production land (70%), followed by orchards and vineyards (29%). The average depth was 0.44 m in the additional inundation extent. There were no parks expected to become inundated as a result of the Council stopbanks being removed. River Road Reserve may experience some inundation in a larger event; however, the park was small (5600 m^2). During a 100-year event, it was expected that significant areas of the Waimea River Park would become flooded; however, this area was contained between the Council stopbanks. The flooding of the Waimea River Park would not be the result of the Council stopbanks failing. Overall the potential damage the Council stopbanks had on the natural environment was classified as Significant but Recoverable.

Downstream of the Council stopbanks there were no major population centres at serious risk of inundation, although areas of Appleby may have experienced some inundation. However, this was difficult to assess as Appleby was outside of the model domain. Within the HEC-RAS model, Saint Michael's Anglican Church became inundated at a depth of approximately 0.5 m because of the removal of the Council stopbanks. Other community facilities such as the Saints Peter and Paul's Catholic Church did not experience flooding, but inundation was modelled on the property. Appleby Primary School was located downstream of the model domain and may experience flooding as a result of the Council stopbanks removal. The damage to production land was likely to be one of the greatest sources of impact on the community. Given the average depth, the crops were estimated to be affected by a damage factor of 30% (JRC, 2017). The community recover time was assessed as Months because of the damage to the production land.

9.10.2. Main Spring Grove Stopbank Assessed Damage Level

Removing the MSG stopbank led to no additional buildings becoming moderately damaged. Because of this, the MSG stopbank had a minimal impact on residential buildings. Similarly, the MSG stopbank was not expected to affect any critical infrastructure. The removal of the stopbank did allow water to flow across SH6 at a depth less than 0.3 m.

There were no parks or reserves located in or near the additional inundation extent. The additional inundation flooding extent because of the removal of the undocumented stopbank was 0.53 km^2. The majority of the land was used as production land (95%) and was flooded to an average depth of 0.15 m. The expected damage factor in production land at this depth was less than 15%.

The Spring Grove Church of Christ was modelled as exposed to flooding, and the depth was less than 0.10 m. The Spring Grove Hall property experienced minor inundation less than 0.10 m, but the hall itself was not inundated, and the inundation was not the result of the MSG stopbank being removed.

9.10.3. Pitfure Stopbank Assessed Damage Level

Within the model, the removal of the Pitfure stopbank did not cause any additional buildings to become inundated. No critical or major infrastructure was affected by the removal of the stopbank. Within Wakefield Township there was a police station and a fire station. However, both of these emergency facilities were located outside of the inundation extent and thus unaffected by the flooding.

The additional area inundated by the removal of the Pitfure stopbank was 0.06 km^2 and was 80% production land and 20% Orchards and Vineyards. The average depth of the additional inundation was 0.07 m. This depth of inundation was estimated to cause less than 10% damage to the production land.

There were no downstream population centres or features of environmental significance affected by the removal of the Pitfure stopbank. The additional flooding of the Pitfure was also unlikely to affect the capability of emergency response efforts.

9.10.4. Population At Risk

To assess the number of PAR, the depth layers from stopbanks design level events were clipped to depths greater than 0.5 m. The land use was derived from the LCDB and occupancy rates were defined (King & Cousin, 2015) and applied to calculate the approximate population at risk. The occupancy rates and PAR of the stopbanks are shown in Table 9.4

Table 9.4. The Population At Risk due to stopbank removal

		Council Stopbanks		MSG Stopbank		Pitfure Stopbank	
Land cover	**Occupancy Rate (persons/km2)**	**Area (km^2)**	**PAR**	**Area (km^2)**	**PAR**	**Area (km^2)**	**PAR**
Orchard or Vineyard	100	0.38	38.4	-	-	0.001	0.1
Production land	20	1.44	28.7	0.025	0.5	0.015	0.3
Total		**1.82**	**67.1**	**0.02**	**1**	**0.02**	**0**

Potential Loss of Life (PLL) was a subset of the PAR. It was defined as the number of fatalities that would be highly likely due to failure. The PLL was found using the Riskscape human losses output. No loss of life was expected in a worse case event where the flood warning was very short (less than 15 minutes) and the understanding of flood risk was vague for any stopbank scenarios simulated.

9.10.5. Estimated Potential Impact Classifications

The PIC classifications were found using the Assessed Damage Level and the PAR. Where multiple classifications were possible, the PLL was used to determine the PIC. A summary of the Assessed Damage Levels and PAR are shown in Table 9.5.

Table 9.5. Summary of PIC assessment

Stopbank	Residential Houses	Critical Structures	Natural Environment	Community Recovery Time	Assessed Damage	PAR	PLL	PIC
Council	Major	Minimal	Moderate	Moderate	**Major**	**67**	0	**High**
MSG	Minimal	Minimal	Minimal	Minimal	**Minimal**	**1**	0	**Low**
Pitfure	Minimal	Minimal	Minimal	Minimal	**Minimal**	**0**	0	**Low**

The Council stopbanks were found to have the greatest potential to cause damage if removed of the stopbanks investigated. Their removal caused eight residential buildings to become moderately damaged in Riskscape. The damage to the residential buildings led to the Council stopbanks to have a Major Assessed Damage Level. The Council stopbanks were also found to have the highest PAR. The majority of the PAR were based in orchards and vineyards although a significant portion were based in production land. The size of the PAR and the Major assessed damaged level resulted in the Council stopbanks being rated as having a High Potential Impact Classification.

The MSG and Pitfure stopbanks in contrast both were found to the have Low assessed damage classifications. This was because the stopbanks did not affect any residential buildings or critical infrastructure as well as having minimal impacts on the natural environment and community. The primary land cover affected was production land which had a low occupation rate. Because of this the stopbanks did not have a High PAR. The result of the Minimal Assessed Damage Levels and low PARs was that the MSG and Pitfure stopbanks both had Low Potential Impact Classifications.

10. LIMITATIONS

- There were several limitations to this study. These are briefly outlined below:
- Vegetation interferes with LiDAR, allowing fewer points to strike the ground. This results in a coarser topographic map in areas of thick vegetation, such as at the undocumented stopbanks.
- The bathymetry was linearly interpolated between cross-sections (on average 360 m and 510 m apart for the Wairoa/Waimea and the Wai-iti respectively). This may have resulted in features such as weirs being omitted, and a final bathymetry that was smoother than in reality.
- The LCDB did not capture land covers such as roads. Because of this, the roads were classified by the surrounding land cover, which was generally production land. Similarly, within Wakefield and Brightwater, the LCDB did not categorise different urban land zones.
- During the estimation of the ungauged flows, it was assumed that the return turn period was the same in all the catchments and that the event started in all catchments at the same time.
- Council flow data derived from rating curves were treated as fully representative of the actual flows despite the limitations of rating curves.
- Historic flood maps originally created by hand were treated as fully representative of the actual flooding extent during calibration and validation.
- The model was calibrated to the 1983 event using 2016 terrain. However it is likely that the channel bed has degraded over the 30 year period due to gravel extraction which has lowered the channel bed by one metre in some locations.
- To simplify the calibration, the channel roughness was divided into two sections. In an actual river, roughness increases/decreases depending on factors such as slope and grain size
- None the validation flood maps encompassed the Wai-iti River. Because of this, the Wai-iti section of the domain was only calibrated using one historical map.
- Erosion of the stopbanks was not considered. The stopbank were considered as fixed and indestructible. As part of this, possible failure (breaching) of the stopbanks was not considered in the model.
- The model did not account for the effects of human actions such sandbagging or pump stations.
- Rainfall was not considered in the model input. The inclusion of rainfall into the model would have caused additional flow in the river and ponding in some locations.
- Within Riskscape, buildings were not always located where the buildings they actually represented were. Multiple buildings were also often clustered together.
- It was not possible to access the fragility curves behind the Riskscape software. Because of this it was difficult to evaluate the uncertainty of the numbers projected by the software.
- The Riskscape analysis did not consider the impact to assets other than buildings. This meant the Riskscape assessment did not capture the impact to production land or orchards/vineyards which were the most widespread land covers protected by the undocumented stopbanks.

11. DISCUSSION, CONCLUSIONS AND FURTHER RESEARCH

11.1. Discussion

People are increasingly populating areas at risk of flooding. Internationally flooding results in billions of dollars of loss annually, despite the presence of stopbanks aimed at reducing the impact of flooding. Within New Zealand there is uncertainty regarding the number of undocumented stopbanks and their subsequent impact. This study aimed to determine the impact of the undocumented stopbanks on flooding extent and buildings by using the Waimea floodplain as a case example.

Many factors can influence the flooding extent and subsequent losses including the flood-event itself, the number of exposed assets and the vulnerability of these assets. The inundation extent can be reduced by flood protection systems such as stopbanks. Maintenance of these stopbanks is critical to long term performance. Both international and New Zealand guidelines highlight that an essential aspect of maintenance involves identification and repair of damage. These guidelines make recommendations against woody vegetation and cattle being present on stopbanks.

This case study identified that the Council stopbanks were the most uniform in geometry and height as well as having the least observed damage of the stopbanks. The Council stopbanks also cultivated a grass cover that was clear of vegetation.

The undocumented stopbanks in contrast were not known to have a formal maintenance scheme and were found to have larger areas of thick vegetation and greater qualities of surface damage compared to the Council stopbanks. Areas of heavy vegetation were also generally separated from the surrounding production land by a fence which prevented access for maintenance. Where there was a lack of vegetation the stopbank was generally unfenced which allowed stock such as cattle to graze the stopbank and remove vegetation. Cattle had also trampled the cover resulting in surface damage. It is possible that other undocumented stopbanks in New Zealand are in a similarly deteriorated state to those in the Waimea floodplain.

The undocumented stopbanks had undulations which caused overtopping in the model. This led to the inundation of the adjacent land that the stopbanks were originally designed to protect. Simulations that removed the undocumented stopbanks found that there was no significant increase in the inundation extent to either the adjacent land or downstream land. This highlights that the undocumented stopbanks in their present state behave similarly to natural terrain. This indicates that the undocumented stopbanks currently do not significantly reduce the impact of flooding in relation to inundation extent.

Several scenarios investigated the implication of maintaining the undocumented stopbanks. When the undocumented stopbanks were simulated as maintained there was an overall reduction in inundation extent of the adjacent land. This reduction led to a reduction in flooding impact. It should be noted however that the maintenance of one of the stopbanks caused more flow to be contained resulting in a slight increase in inundation downstream.

The Council stopbanks maintained by the TDC had the least amount of woody vegetation and surface damage of the stopbanks assessed. The Council stopbanks were found to prevent the greatest area from inundation of all the stopbanks investigated. The Council stopbanks did not overtop in any of the return periods simulated.

The simulated removal of the Council stopbanks found that their removal led to significantly more area to be inundated. The flood extent assessment demonstrated that when the stopbanks were maintained they were better able to protect against inundation in the adjacent land. This reduced the risk of flooding in protected areas.

This raises questions about the cost of maintaining the undocumented stopbanks versus the cost of flood losses. Research pertaining to European and US stopbanks found that stopbank maintenance was achieved at relatively low financial cost while (re)construction of stopbanks was often relatively expensive.

The cost of flood losses is related to several factors including the amount and types of assets exposed. A Riskscape analysis was used to assess impact of the undocumented stopbanks. This analysis was limited to building assets. The Council stopbanks reduced the number of moderately damaged houses by 28%. The reduction of moderately damaged buildings caused a net reduction in reinstallment costs of $1.6M in a 100-year event. The current undocumented stopbanks did not reduce the inundation extent significantly and therefore there was no significant reduction in the number of moderately damaged buildings or total reinstallment costs. When undocumented stopbanks were simulated as maintained the damage states and reinstallment costs were not significantly reduced despite the reduction in inundation area. The condition of the undocumented stopbanks did not influence the reinstallment costs. This was because the analysis was limited to damage to buildings and there were very few buildings near the stopbanks. The reason for the low density of buildings was that the undocumented stopbanks primary protected production land.

The undocumented stopbanks in this study were originally built in the 1950s and 1980s to protect the surrounding production land. Using the LCDB it was possible to see how the land cover use has changed between 1996 and 2012. The greatest decline in the land-use area was to production land. The greatest increase in land-use area was to orchards and vineyards. A landowner commented during the condition assessment that three years prior they had converted their paddocks to produce hops.

Table 11.1. Comparison of land cover usage within the domain between 1996 and 2012

Land cover	**1996 Area (km^2)**	**1996 Area Percentage (%)**	**2012 Area (km^2)**	**2012 Area Percentage (%)**	**Percent Change**
Orchard/Vineyard	12.65	16.0	15.65	19.8	+3.8
Township	1.93	2.4	3.30	4.2	+1.7
Production Land	54.17	68.5	49.55	62.6	-5.8
Other	10.37	13.11	10.62	13.42	+0.31
Total	127				

A recent study (NIWA, 2019b) found that the type of land most exposed to flood hazard in New Zealand was that of production land (15,190 km^2). This was nearly three-times the area exposed to natural or undeveloped land covers (8,730 km^2). Damage to crops is difficult to quantify because the loss depends on several factors such as seasonality, crop value, and crop type. An increase in the number of higher value vineyards and orchards (compared to paddocks) would represent a higher loss if damaged during a flood. If production land continues to be developed into vineyards and orchards without the development of additional flood protection measures, potential losses may increase in the future.

Several of the limitations of this study could be addressed by gathering greater quantities of data relating to flood maps, undocumented stopbank properties and gauges.

The number of flood maps held by the TDC was limited and many did not cover the entire domain of this study. The older flood maps were created by hand and thus were imperfect replications of the actual flood extents. The model was therefore calibrated and validated against older, approximated maps that did not cover the entirety of the domain. This limitation could be addressed by developing flood maps from recent flood events. This would allow flood models to be calibrated and validated more accurately, and result in a more robust and realistic model that would better predict the impact of the undocumented stopbanks.

At the outset of this study there was no data available regarding the geometry or location of undocumented stopbanks. The lack of information regarding undocumented stopbanks is an issue in New Zealand and internationally (ICOLD, 2018). The lack of information in New Zealand may be due to the lack of a standardised approach to stopbank management. Nationwide there is an absence of information on documented and undocumented stopbanks regarding location, geometry, age, material properties and design capacity (Blake et al., 2018). The geometry of the stopbanks was represented using the terrain generated by LiDAR because of lack of information. However, vegetation on the undocumented stopbanks creates coarser topographic maps. Increased stopbank information regarding locations and geometry could result in a better representation of undocumented stopbanks.

Increased data on the material properties of the stopbank may also be able to address the limitation of breaching which was not considered in this study. A German study found that the main cause of breaching was due to external erosion that was predominantly the result of overtopping (Horlacher et al., 2007). Given the velocity and duration of flooding and the cover of the undocumented stopbanks, there is evidence to suggest that erosion is likely to occur (Hewlett et al., 1987). The consideration of stopbank breaching scenarios requires knowledge of material properties. A breaching scenario may have led to a different inundation extent and resulted in a more accurate prediction of the impact of the stopbanks. Applying and cataloguing the properties of regional stopbanks would be beneficial to flood risk management nationwide.

There are many catchments internationally that are either poorly gauged or ungauged. Within the study area there were nine ungauged basins where it was necessary to approximate the flows. There are many accepted methods for estimating the peak flow. These methods however are only approximations of the actual flow and therefore carry some level of uncertainty. This could be reduced by placing gauging stations at the ungauged catchments. Flows generated from rating curves however also carry a level of uncertainty. Gauging would represent an improvement for determining the model flow inputs and therefore an improvement in model accuracy and impact assessment.

Overall, increased data would enable a more accurate assessment of the impact of undocumented stopbanks on flood risk

11.2. Conclusions

This section presents a summary of key findings from the study:

- The Council stopbanks were found to be well maintained and had a uniform grass cover along the 4 km surveyed. Several structures were also observed in the Council stopbanks including six pump stations and two power poles that had been built into the stopbank.
- The MSG stopbank was covered approximately in half by grasses and half heavy vegetation. Cattle were seen grazing on the stopbank. Surface damage was observed in these areas.
- The western side of the Pitfure stopbank was characterised by woody vegetation that in several cases had rotted, leaving large voids in the stopbank. The eastern Pitfure stopbank was predominately free of woody vegetation but in places had undulations and been scoured. This may have been the result of the channel straightening that had occurred while constructing the western Pitfure stopbank.
- The Wairoa Confluence stopbank had a cover of woody vegetation cover and newly planted growth as part of the Waimea River Park Management Plan. At the access roads to the river, large decreases in crest height were observed. The Wai-iti Confluence stopbank which was on private land and had an impenetrable cover of gorse and broom for the majority of its length.

- The Council stopbanks were both the largest and had the most uniform cross-section of the stopbanks. The average height of the stopbanks was 2.6 m compared to the surrounding adjacent land and designed to route a 50-year event with 0.6 m of freeboard. The slope of the stopbanks generally was a 2.1H: 1V ratio on both sides, this was within the maximum design levels recommended by the Bay of Plenty guidelines (BoPRC, 2014). The crest of the stopbank was typically 3.2 m wide and able to accommodate standard vehicle passage along the top of the stopbank.
- The MSG stopbank had a fluctuating height. The downstream section had an average height of 1.2 m with crest undulations of ±0.2 m. The upstream extent of the stopbank was higher with an average waterside height of 2.3 m. The slopes of the stopbank were between 2.7 H: 1V and 2H: 1V, which were within the recommended limits set by the BoP guidelines. (BoPRC, 2014).
- The Pitfure crest height was the lowest of the stopbanks and was assessed at approximately 0.9 m high. It had undulations of ± 0.5 m. The western side of the Pitfure stopbank was characterised by large woody vegetation. Along the Pitfure Stream in many places, the stopbank was a sheer face.
- The Confluence stopbank fluctuated from 0.8 m high at the upstream end of the Wairoa to 3 m high toward at the downstream end. The Wai-iti side of the Confluence stopbank was approximately 1.5 m high.
- A flood routing model was created using HEC-RAS 2-D software. For the flow inputs, it was found that the ungauged tributaries contributed the majority of the flow to the Wai-iti. The most suitable and comprehensive method to estimate this was determined to be the TM61 method developed by the Ministry of Works (1984).
- The HEC-RAS model was calibrated against historic flood extents and optimised using roughness values. The roughness values of the floodplain were first calibrated, followed by the river channels. The model was determined to be relatively insensitive to the floodplain roughness values. The final roughness values for the channel were 0.05 and 0.03 for the Wai-iti/Pitfure and the Wairoa/Waimea respectively.
- The inundation extent caused by removing and maintaining the stopbanks was determined.
  - The Council stopbanks did not overtop in a 100-year event and were able to maintain one metre of freeboard. Some flow was able to bypass the stopbank by flowing around the top of the Wai-iti side. The Council stopbanks were shown to prevent 4.2 km^2 and 5.8 km^2 of land from inundation in 5 and 100-year events respectively.
  - The MSG stopbank overtopped in multiple locations due to undulations in crest level. The land cover where there was overtopping composed of impenetrable vegetation. The overtopping caused flooding of several private properties that were predominantly surrounded by production land. The removal of the stopbank led to up to an additional 0.50

km^2 of inundation to occur. The difference in the inundation area between when the stopbank was present and removed decreased as the return period increased.

- The Pitfure stopbank overtopped in several places due to undulations in the crest level. These undulations were seen during the condition assessment. When the stopbank was removed, the flooded area increased by 0.09 km^2. The reason for the minimal increase in inundation extent was that the undulations allowed flow to pass through the stopbank.
- The Confluence stopbank overtopped in two locations on the Wai-iti side. However, the greatest source of water into the confluence area was from flow around the upstream end of the Wai-iti stopbank. Together these caused widespread inundation in the confluence area. The Confluence stopbank protected approximately 0.08 km^2 in a 5-year event. The difference in the inundation area between when the stopbank was present and removed decreased as the return period increased. The confluence area itself composed of a mixture of orchard/vineyard and production land.
- The modelled maintained MSG stopbank despite having its undulations removed was still overtopped in events greater than 50 years. The area protected increased from 0.42 to 0.77 km^2 in 5 to 100-year events respectively. This land was categorised as 'high producing exotic grassland' in the LCDB. The modelled maintained MSG stopbank increased flow contained in the Wai-iti which caused additional areas to be inundated at an airstrip at the downstream section of the stopbank.
- The model of a maintained Pitfure stopbank should prevent the inundation of up to 0.19 km^2. The stopbank did not overtop in any of the return periods tested. The area protected was mostly used as production land with one small area of orchard/vineyard.

- Riskscape was also used to quantify the damage to buildings. A limitation of the study was that damage to non-building assets was not captured and the primary area protected by the undocumented stopbanks was production land which has a low building density.
  - During the 100-year event with all stopbanks present, 30 properties were estimated to be moderately damaged. The majority of these were located near Brightwater Township as a result of the Wairoa flooding. Because of the flooding, 26 properties were predicted to be unable to be occupied for a week, and nine could not be occupied for over six months. The total building reinstallment cost was approximated as \$8.7M in a 100-year event with a significant portion of the cost arising from stock damage to buildings at Fonterra Brightwater.
  - The model showed that the Council stopbanks protected up to 12 properties, mostly west of the stopbanks, from moderate damage. This reduced the overall reinstallment cost by \$1.6M.
  - When the MSG stopbank was modelled as removed there was no increase in the number of moderately damaged buildings. This was because the MSG stopbank had undulations in its

crest. The land adjacent was also primarily production land with a low density of buildings, lowering the potential for building damage. Because of these factors, the removal of the stopbank led to a relatively modest increase of $26,000 in reinstallment costs.

- When the Pitfure stopbank was modelled as removed there was no increase in the number of buildings affected or the severity of building damage compared to when the stopbank was present. The reasons for this were the same as the MSG stopbank; firstly, the low density of buildings in the adjacent land and secondly, undulations in the current stopbank. Because there was no significant increase in the building damage, the overall reinstallment cost was essentially the same as when the stopbank was present.
- The Confluence stopbank currently protects one property from becoming severely damaged. The removal of the Confluence stopbank allowed increased flow into the confluence area, which increased the velocity and duration of flooding that the building was exposed to. The additional costs to this building were estimated in the Riskscape model as $2,500.
- The maintained MSG stopbank prevented several buildings near Telenius Road from being exposed to flooding. The lack of flooding in the area adjacent to the stopbank however meant increased flow was contained in the Wai-iti. The additional flow resulted in an increase in moderately and lightly damaged buildings downstream. As a result, the reinstallment costs did not decrease significantly from the present scenario. The cost increased by $50,000 during a 10-year event.
- The maintained Pitfure stopbank prevented one property from moderate damage. Similar to the removal scenario, the lack of impact was due to the lack of properties near the Pitfure stopbank. The maintained Pitfure stopbank prevented on average $150,000 of reinstallment costs. The saving derived from two buildings clustered closely together in the Riskscape model.

- The NZSOLD method to classify the impact of dams was applied to determine a Potential Impact Classification for the stopbanks. The Council stopbanks had a High PIC as they were estimated to cause 8 residential buildings to become uninhabitable and the PAR was 67 people. The MSG and Pitfure stopbanks both had Minimal Assessed Damage Levels as they did not affect any residential buildings or critical infrastructure as well as having a Minimal impact on the natural environment and community recovery time. These stopbanks also had a low PAR. The PAR coupled with the Assessed Damage Level resulted in the MSG and Pitfure stopbanks both having Low PICs
- This study found, through flood modelling, that the undocumented stopbanks do not currently have a significant impact on inundation extent or building impact. However, if the undocumented stopbanks were maintained they could reduce the inundation extent. This may lead to fewer flood losses, particularly to production land as well as orchards and vineyards.

11.3. Further Research

Advances in technology have allowed large asset networks to be documented. This plays an important role in the collation and distribution of such data. At the beginning this project it became apparent that, although there was informal information on the locations and extent of the undocumented stopbanks, there was no systematic documentation. By formally documenting the location of the stopbanks decision making on flood risk management can be better informed. In addition to a detailed register of the undocumented stopbanks, use of response teams to survey the inundation extent during a flood would be of benefit to the creation of better flood maps. Flood maps raise awareness of the flood hazard and locate areas at risk. Together these tools may also be able to identify areas where there is a risk of breaching and ensure appropriate action is taken before a significant event. The creation of more modern flood maps could allow flood models to be calibrated to the most recent flood events, improving their ability to simulate the current flood impact.

The flood model could be improved with further research into the likelihood of erosion and breach scenarios. The model currently does not include the effects of erosion. It is believed from the flood model that this is likely to occur in some instances. Development of knowledge regarding material properties for example would allow the model to more accurately represent the actual flooding extent if a serious flood event were to occur.

This study indicated that most of the building damage in the study catchment appeared to be located in Brightwater Township. The cause of this damage was due to water overflowing at Factory Road from the Wairoa River. This study did not consider the creation of new stopbanks. Further research could be directed at the impact of creating new stopbanks at this location and other areas where the rivers overtop. The potential for this research would allow the benefits of new stopbanks to be investigated before their inception. This, again would allow the making of more informed flood risk decisions.

A critical area of further research that would benefit this study would be a quantification of the damage to production land and orchards/vineyards because the primary objective of these stopbanks is to protect these land covers. Further research in this area would allow a core component of the stopbank impact assessment to be developed. It is difficult to quantify the impact of the stopbanks without this aspect and inclusion would result in a much more accurate estimate of the losses associated with each flood.

It is hoped that this case study on the impacts of the undocumented stopbanks on flood extent within the Waimea floodplain will be utilised to better understand the effects of unregistered and privately-owned stopbanks throughout New Zealand.

12. REFERENCES

AAM. (2017). *Existing Tasman and Nelson LiDAR conversion to NZVD2016*. Napier, N.Z:

Aggarwal, A. (2016, 2016/07/03). Exposure, hazard and risk mapping during a flood event using open source geospatial technology. *Geomatics, Natural Hazards and Risk, 7*(4), 1426-1441. https://doi.org/10.1080/19475705.2015.1069408

Alcrudo, F. (2004). *A state of the art review on mathematical modelling of flood propagation*. Zaragoza, Spain: University of Zaragoza. Retrieved from http://www.impact-project.net

Association of State Floodplain Managers. (2013). *Flood mapping for the nation: A cost analysis for the nation's flood map inventory*. Retrieved from https://www.floods.org

Bates, P. D., & De Roo, A. P. J. (2000). A simple raster-based model for flood inundation simulation. *Journal of Hydrology, 236*(1), 54-77. https://doi.org/10.1016/S0022-1694(00)00278-X

Bay of Plenty Regional Council. (2012). *Hydrological and hydraulic guidelines*. Whakatāne, N.Z: Retrieved from https://www.boprc.govt.nz

Bay of Plenty Regional Council. (2014). *Stopbank design and construction guidelines*. Whakatāne, N.Z: Retrieved from https://www.boprc.govt.nz

Betsholtz, A., & Nordlöf, B. (2017). *Potentials and limitations of 1D, 2D and coupled 1D-2D flood modelling in HEC-RAS*, Lund University, Lund, Sweden). Retrieved from https://www.lu.se

Blake, D. M., Pascoal, E., Rodger, M., Crawford-Flett, K., Wilson, M., & Wotherspoon, L. (2018). *Mapping New Zealand's stopbank network: a standardised nationwide inventory*. presented at the International Conference on GIS and Geoinformation Zoning for Disaster Mitigation, Auckland, N.Z.

BMT. (2017). *Benchmarking Results TUFLOW Products 2017-09*. Australia: Retrieved from https://www.tuflow.com

Brakenridge, R., Andersona, E., Nghiemb, S., Caquard, S., & Shabaneh, T. (2003, 01/01). *Flood warnings flood disaster assessments and flood hazard reduction: The roles of orbital remote sensing*. presented at the International Symposium on Remote Sensing of Environment, Vancouver, Canada. Retrieved from http://hdl.handle.net/2014/38308

Brennan, S. (2015). Local authority liability for flooding: Where should loss fall? *Victoria University of Wellington Law Review, 46*(1), 85-113. https://doi.org/10.26686/vuwlr.v46i1.4933

Centre for Research on the Epidemiology of Disasters. (2019, October 14 2019). Emergency Events Database

Chaudhry, M. H. (2008). *Open-channel flow* (2nd ed.). New York, U.S: Springer.

Construction Industry Research and Information Association, United States Army Corps of Engineers, & Ministère de l'Ecologie du Développement durable et de l'Energie. (2013). *The international levee handbook*.

Cook, A., & Merwade, V. (2009, 2009/10/20/). Effect of topographic data, geometric configuration and modeling approach on flood inundation mapping. *Journal of Hydrology, 377*(1), 131-142. https://doi.org/https://doi.org/10.1016/j.jhydrol.2009.08.015

Coumou, D., & Rahmstorf, S. (2012). A decade of weather extremes. *Nature Climate Change, 2*(7), 491-496. https://doi.org/10.1038/nclimate1452

Department of Conservation. (1977). *Reserves act*. Wellington, N.Z: Retrieved from http://www.legislation.govt.nz

Department of Internal Affairs. (2002). *Local Government Act*. Wellington, N.Z: New Zealand Law Society. Retrieved from http://www.legislation.govt.nz

El Kadi Abderrezzak, K., Paquier, A., & Mignot, E. (2009). Modelling flash flood propagation in urban areas using a two-dimensional numerical model. *Natural Hazards, 50*(3), 433-460. https://doi.org/10.1007/s11069-008-9300-0

Environment Agency. (2013). *Benchmarking the latest generation of 2D hydraulic modelling packages*. Retrieved from https://www.gov.uk

Ericksen, N. (1986). *Creating flood disasters?: New Zealand's need for a new approach to urban flood hazard* (Vol. no. 77). Wellington, N.Z: Ministry of Works.

Farooq, M., Shafique, M., & Khattak, M. S. (2019). Flood hazard assessment and mapping of River Swat using HEC-RAS 2D model and high-resolution 12-m TanDEM-X DEM (WorldDEM). *Natural Hazards, 97*(2), 477-492. https://doi.org/10.1007/s11069-019-03638-9

Federal Emergency Management Agency. (2005). *Conduits through embankment dams*. Colorado, U.S: Retrieved from https://www.hsdl.org

Federal Emergency Management Agency. (2009). *Mapping the zone: improving flood map accuracy*. https://doi.org/10.17226/12573

French, R. H. (1985). *Open-channel hydraulics*. New York, U.S: McGraw-Hill.

Gharbi, M., Soualmia, A., Dartus, D., & Masbernat, L. (2016). Comparison of 1D and 2D hydraulic models for floods simulation on the medjerda riverin tunisia. *Journal of Materials and Environmental Science, 7*(8), 3017-3026.

Golder Associates Ltd. and Associated Engineering Ltd. (2003). *Dike design and construction guide - Best management practices for British Columbia*. Retrieved from https://www2.gov.bc.ca/

Griffith, G. (2018, July 1, 2018).

Hardoy, J., Mitlin, D., & Satterthwaite, D. (2001). *Environmental problems in an urbanizing world: Finding solutions in cities in Africa, Asia and Latin America*. https://doi.org/10.4324/9781315071732

Hewlett, H. W. M., Boorman, L. A., & Bramley, M. E. (1987). *Design of reinforced grass waterways* (Vol. 116). London, U.K: CIRIA.

Hicks, D. M., & Mason, P. D. (1991). *Roughness characteristics of New Zealand rivers*. Wellington, N.Z: Water Resources Survey, DSIR Marine and Freshwater.

Higgins, M. (2018, January 17 2019). Wai-iti

Horlacher, H. B., Heyer, T., Carstensen, D., Bielagk, U., Bielitz, E., & Müller, U. (2007, 7–11 May 2007). *Analysis of dyke breaks during the 2002 flood in Saxony/Germany.* Paper presented at the 2nd Lake Abaya Research Symposium, Arba Minch, Ethiopia.

Horritt, M. S., & Bates, P. D. (2001). Predicting floodplain inundation: raster-based modelling versus the finite-element approach. *Hydrological Processes, 15*(5), 825-842. https://doi.org/10.1002/hyp.188

Huizinga, J., de Moel, H., & Szewczyk, W. (2017). *Global flood depth-damage functions* (978-92-79-67781-6). Sevilla, Spain: Joint Research Centre. https://doi.org/10.2760/16510

Hunter, N. M., Bates, P. D., Neelz, S., Pender, G., Villanueva, I., Wright, N. G., . . . Mason, D. C. (2008). Benchmarking 2D hydraulic models for urban flooding. *Proceedings of the Institution of Civil Engineers: Water Management, 161*(1), 13-30. https://doi.org/10.1680/wama.2008.161.1.13

Insurance Council of New Zealand. (2014). *Protecting New Zealand from natural hazards*. Wellington, N.Z: Retrieved from https://www.icnz.org.nz

Insurance Council of New Zealand. (2019). Cost of natural disasters. Retrieved2019, from https://www.icnz.org.nz/natural-disasters/cost-of-natural-disasters/

International Commission on Large Dams European Club. (2018). *European and US levees and flood defences: characteristics, risks and governance*. https://doi.org/10.24346/cfbr_eurcold2018

Khan, S. I., Hong, Y., Wang, J., Yilmaz, K. K., Gourley, J. J., Adler, R. F., . . . Irwin, D. (2011). Satellite remote sensing and hydrologic modeling for flood inundation mapping in Lake Victoria basin: Implications for hydrologic prediction in ungauged basins. *IEEE Transactions on Geoscience and Remote Sensing, 49*(1), 85-95. https://doi.org/10.1109/TGRS.2010.2057513

King, A., & Cousin, W. (2015). *Temporal and building use implications on earthquake risk evaluation*. presented at the New Zealand Society for Earthquake Engineering Technical Conference, Rotorua, N.Z. Retrieved from https://www.nzsee.org.nz

Klemas, V. (2015). Remote sensing of floods and flood-prone areas: An overview. *Journal of Coastal Research, 31*(4), 1005-1013. https://doi.org/10.2112/JCOASTRES-D-14-00160.1

Knight, D. W., & Shamseldin, A. Y. (2006). *River basin modelling for flood risk mitigation* (1st ed.). https://doi.org/10.1201/9780367803209

Kreibich, H., Piroth, K., Seifert, I., Maiwald, H., Kunert, U., Schwarz, J., . . . Thieken, A. H. (2009). Is flow velocity a significant parameter in flood damage modelling? *Natural Hazards and Earth System Science, 9*(5), 1679-1692. https://doi.org/10.5194/nhess-9-1679-2009

Kundzewicz, Z. W., Su, B. D., Wang, Y. J., Wang, G. J., Wang, G. F., Huang, J. L., & Jiang, T. (2019). Flood risk in a range of spatial perspectives - from global to local scales. *NATURAL HAZARDS AND EARTH SYSTEM SCIENCES, 19*(7), 1319-1328. https://doi.org/10.5194/nhess-19-1319-2019

Landcare Research. (2010). *NZDEM South Island 25 metre*. https://doi.org/10.7931/L1R94
Landcare Research. (2015a). Land Cover Database version 4.1
Landcare Research. (2015b). *New Zealand land cover database classes at version 4.1*.Landcare Research New Zealand Ltd. https://doi.org/10.7931/L17H3
Legates, D. R., & McCabe, G. J. (1999). Evaluating the use of "goodness-of-fit" Measures in hydrologic and hydroclimatic model validation. *Water Resources Research, 35*(1), 233-241. https://doi.org/10.1029/1998WR900018
Lighthill, M. J., & Whitham, G. B. (1955). On kinematic waves. I. flood movement in long rivers. *Proceedings of the Royal Society A, 229*(1178), 281-316. https://doi.org/10.1098/rspa.1955.0088
Macara, G. R. (2016). *The climate and weather of Nelson and Tasman* (2nd ed.). Retrieved from https://www.niwa.co.nz
Manaaki Whenua. (2019). *S-map*. Retrieved from https://smap.landcareresearch.co.nz/
Miller, J. E. (1984). Basic concepts of kinematic-wave models. *Geological Survey Professional Paper*(1302).
Ministry for the Environment New Zealand. (1991). *Resource management act*. Wellington, N.Z: Retrieved from http://www.legislation.govt.nz
Ministry for the Environment New Zealand. (2010). *Tools for estimating the effects of climate change on flood flow*. Retrieved from https://www.mfe.govt.nz/
Ministry for the Environment New Zealand, Flood Risk Management New Zealand, & River Control Review Steering Group. (2008). *Meeting the challenges of future flooding in New Zealand*. Retrieved from https://www.mfe.govt.nz
Ministry of Works. (1984). *A method for estimating design peak discharge: technical memorandum no. 61*. Wellington, N.Z: Ministry of Works,.
Munson, B. R., Young, D. F., & Okiishi, T. H. (2010). *Fundamentals of fluid mechanics* (6th ed.). New York, U.S: Wiley.
Nash, J. E., & Sutcliffe, J. V. (1970). River flow forecasting through conceptual models part I — A discussion of principles. *Journal of Hydrology, 10*(3), 282-290. https://doi.org/10.1016/0022-1694(70)90255-6
National Institute of Water and Atmospheric Research. (2009). July 1983 Upper South Island Flooding Retrieved from https://hwe.niwa.co.nz/event/July_1983_Upper_South_Island_Flooding
National Institute of Water and Atmospheric Research. (2016). *Climate change impacts on agricultural water resources and flooding* (2016114CH: 71).National Institute of Water and Atmospheric research. Retrieved from https://www.mpi.govt.nz
National Institute of Water and Atmospheric Research. (2018a, March 5 2018). *Climate summary for February 2018*.National Institute of Water and Atmospheric Research,. Retrieved from https://www.niwa.co.nz
National Institute of Water and Atmospheric Research. (2018b). High Intensity Rainfall Design System
National Institute of Water and Atmospheric Research. (2019a). *2018 annual climate summary*.National Institute of Water and Atmospheric Research,. Retrieved from https://www.niwa.co.nz
National Institute of Water and Atmospheric Research. (2019b). *New Zealand fluvial and pluvial flood exposure*.National Institute of Water and Atmospheric Research.
Nelson City Council, & Tasman District Council. (2019). *Top of the south maps*. Retrieved from https://www.topofthesouthmaps.co.nz
New Zealand Institute of Economic Research. (2004). *Economic impacts on New Zealand of climate change related extreme events*. Wellington, N.Z: Retrieved from https://www.mfe.govt.nz
New Zealand Society on Large Dams, & Institution of Professional Engineers New Zealand. (2015). *New Zealand dam safety guidelines* (1st ed.). Wellington, New Zealand.
Organization of American States. (1991). *Primer on Natural Hazard Management in Integrated Regional Development Planning*. Washington, D.C.: Retrieved from http://www.oas.org
Oudin, L., Andréassian, V., Perrin, C., Michel, C., & Moine, N. L. (2008). Spatial proximity, physical similarity, regression and ungaged catchments: A comparison of regionalization approaches based on 913 French catchments. *Water Resources Research, 44*(3), 2-15. https://doi.org/10.1029/2007WR006240

Park, K., & Lee, M. H. (2019). The development and application of the urban flood risk assessment model for reflecting upon urban planning elements. *Water Journal, 11*(5), 920. https://doi.org/10.3390/w11050920

Paudel, M., Roman, R., & Prichard, J. (2016). *A comparative study of HEC-RAS 2D, TUFLOW, & Mike 21*. California, USA: Wood Rodgers. Retrieved from http://www.floods.org

Pearson, C. P., & McKerchar, A. I. (1989). Flood estimation - a revised design procedure. *Transactions of the Institution of Professional Engineers New Zealand: Civil Engineering Section, 16*(2), 59-65.

Quayle, A. M., Pointer, M. W., & Challands, N. (1983). *The Marlborough - Nelson Bays floods on 8-10 July 1983*. Wellington, N.Z: New Zealand Meteorological Service Retrieved from http://docs.niwa.co.nz

Rahman, M. (2006). *Digital Surface Model (DSM) construction and flood hazard simulation for development plans in Naga City, Philippines*, Enschede, the Netherlands). Retrieved from https://www.itc.nl/

Razavi, T., & Coulibaly, P. (2013). Streamflow prediction in ungauged basins: Review of regionalization methods. *Journal of Hydrologic Engineering, 18*(8), 958-975. https://doi.org/10.1061/(ASCE)HE.1943-5584.0000690

Rouse, H. (2012). *Flood risk management research in New Zealand: Where are we, and where are we going?* (978-0-478-19883-6). Lower Hutt, N.Z: GNS Science.

Salami, R. O., von Meding, J. K., & Giggins, H. (2017). Vulnerability of human settlements to flood risk in the core area of Ibadan metropolis, Nigeria. *Jamba: Journal of Disaster Risk Studies, 9*(1), 371-371. https://doi.org/10.4102/jamba.v9i1.371

Schmidt, J., Matcham, I., Reese, S., King, A., Bell, R., Henderson, R., . . . Heron, D. (2011, 2011/09/01). Quantitative multi-risk analysis for natural hazards: a framework for multi-risk modelling. *Natural Hazards, 58*(3), 1169-1192. https://doi.org/10.1007/s11069-011-9721-z

Sinclair Knight Merz. (2013). *Brightwater - Wakefield flood hazard mapping*. Wellington: Retrieved from https://www.tasman.govt.nz

Singh, V. P. (2001). Kinematic wave modelling in water resources: a historical perspective. *Hydrological Processes, 15*(4), 671-706. https://doi.org/10.1002/hyp.99

Sivapalan, M., Takeuchi, K., Franks, S. W., Gupta, V. K., Karambiri, H., Lakshmi, V., . . . Zehe, E. (2003, 2003/12/01). IAHS Decade on predictions in ungauged basins, 2003–2012: Shaping an exciting future for the hydrological sciences. *Hydrological Sciences Journal, 48*(6), 857-880. https://doi.org/10.1623/hysj.48.6.857.51421

Streeter, V. L., Wylie, E. B., & Bedford, K. W. (1998). *Fluid mechanics* (9th ed.). Boston, U.S: WCB/McGraw Hill.

Swift, B. (2014). *Comparison and utilization of 1D, 2D and 3D hydraulic models on a complex diversion structure*. USA: HDR. Retrieved from https://www.floods.org/

Tasman District Council. (1983, December 13 1983). *Wai-iti at Belgrove gauge notes*.

Tasman District Council. (2010). *Waimea river park management plan*. Richmond, N.Z: Retrieved from https://www.tasman.govt.nz

Tayefi, V., Lane, S. N., Hardy, R. J., & Yu, D. (2007). A comparison of one- and two-dimensional approaches to modelling flood inundation over complex upland floodplains. *Hydrological Processes, 21*(23), 3190-3202. https://doi.org/10.1002/hyp.6523

Teng, J., Jakeman, A. J., Vaze, J., Croke, B., Dutta, D., & Kim, S. (2017, 04/01). Flood inundation modelling: A review of methods, recent advances and uncertainty analysis. *90*, 201-216. https://doi.org/10.1016/j.envsoft.2017.01.006

Tomlinson, A. I. (1978). Storm rainfalls. *Journal of Hydrology, 17*(2), 57-77.

Ullah, S., Farooq, M., Sarwar, T., Tareen, M. J., & Wahid, M. A. (2016). Flood modelling and simulations using hydrodynamic model and ASTER DEM—A case study of Kalpani River. *Arabian Journal of Geosciences, 9*(6), 1-11. https://doi.org/10.1007/s12517-016-2457-z

United Nations Office for Disaster Risk Reduction. (2015). The human cost of weather-related disasters 1995-2015.

United States Army Corps of Engineers. (1992). *Design, construction, and maintenance of relief wells* (9780872629554). Washington, D.C.:

United States Army Corps of Engineers. (2000). *Design and construction of levees*. Washington, D.C.: Retrieved from https://www.publications.usace.army.mil
United States Army Corps of Engineers. (2008). Flood damage reduction segment/system inspection report
United States Army Corps of Engineers. (2016a). *HEC-RAS river analysis system: 2D modeling user's manual*. Retrieved from https://www.hec.usace.army.mil
United States Army Corps of Engineers. (2016b). *HEC-RAS river analysis system: Hydraulic reference manual*. Retrieved from https://www.hec.usace.army.mil
United States Army Corps of Engineers. (2018). *Benchmarking of the HEC-RAS two-dimensional hydraulic modeling capabilities*. Washington, D.C.: Retrieved from https://www.hec.usace.army.mil
Walrond, C. (2015). Nelson places - Richmond and the Waimea Plains. Retrieved April 22, 2018, from https://teara.govt.nz
Wanielista, M. P., Kersten, R., & Ealgin, R. (1997). *Hydrology: water quantity and quality control* (2nd ed.). New York, U.S: John Wiley & Sons.
Whitehouse, P. (2014). *Flood risk management*. Retrieved from https://www.waternz.org.nz/
Xichun, J., Ruilan, W., Hao, D., Wei, Z., Zhiwei, L., & Peitong, C. (2017, 2017;2018;). *A neural network method for risk assessment and real-time early warning of mountain flood geological disaster*. Paper presented at the International Symposium on Intelligent Signal Processing and Communication Systems, Xiamen, China.
Zemansky, G., Hong, Y. T., Rose, J., Song, S., & Thomas, J. (2012). Assessing the effects of climate change on water resources: the Waimea Plains. *Journal of Hydrology (New Zealand), 51*(1), 45-61.

Appendix A: CONDITION ASSESSMENT CROSS-SECTIONS

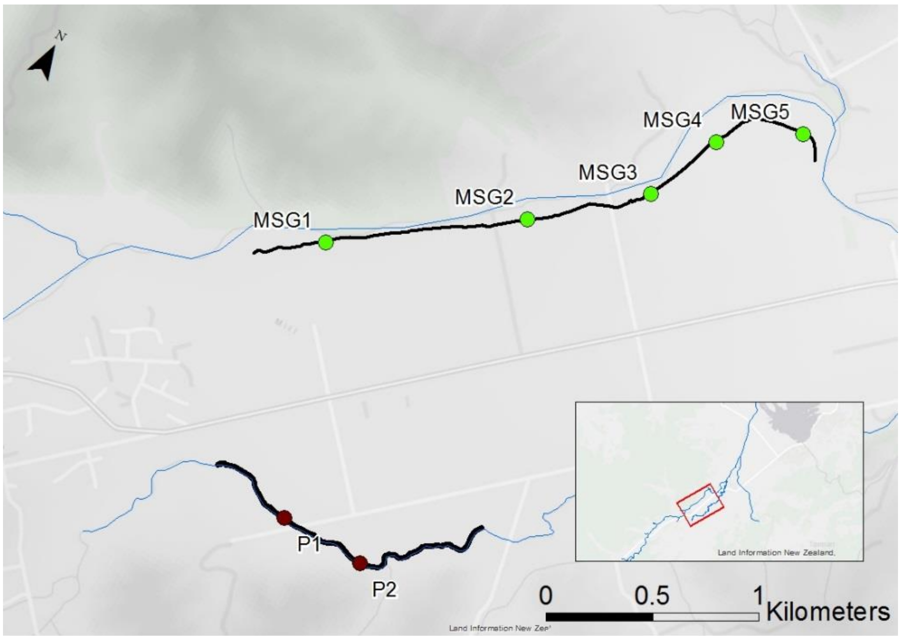


Figure A.1. Locations of cross-sections taken along the MSG and Pitfure stopbanks

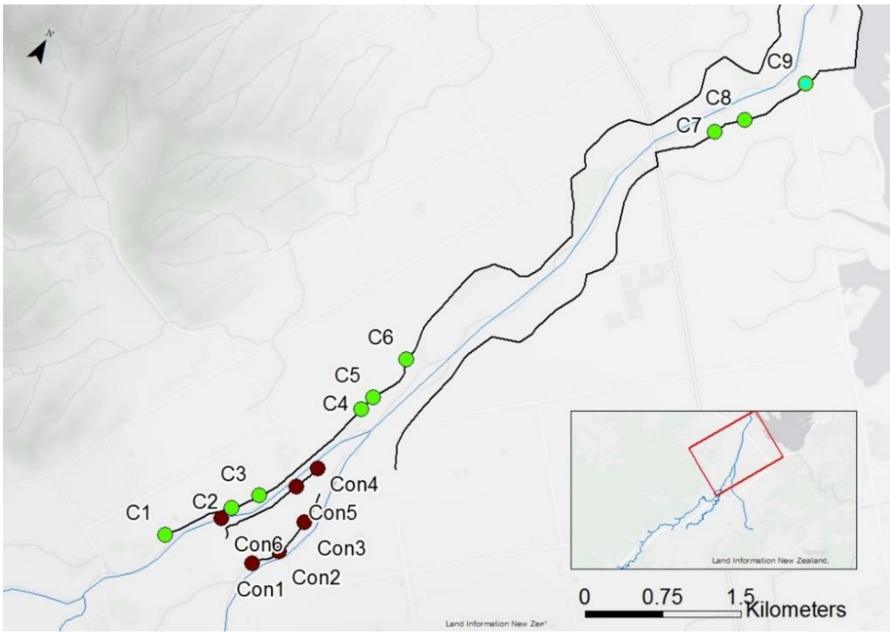


Figure A.2. Locations of cross-sections taken along the Council and Confluence stopbanks

Table A.1. Cross-section locations and measurements

Status	Indictor	River	Stopbank	Crest Width (m)	Upstream Height (m)	Upstream Slope (°)	Upstream Cover	Downstream Height (m)	Downstream Slope (°)	Downstream Cover	Structures	Comments	Easting NZGD 2000	Northing NZGD 2000
Undocumented	P1	Pitfure	Pitfure	1	2.2	30	Wooded	0.7	22	Grasses, Tree			1605102	5416345
Undocumented	P2	Pitfure	Pitfure	1.2	2.9	34	Grasses	1	31	Grasses	Fence at Downstream toe	Upstream: 1.1 m vertical, 1.1 m sloping	1605517	5416338
Undocumented	MSG1	Wai-iti	MSG	0.5	2.6	16	Grasses	1.4	20	Grasses - Patchy		Upstream: 1.4 m vertical, 1.5 m sloping	1604624	5417565
Undocumented	MSG2	Wai-iti	MSG	2	2.3	22	Grasses	1.5	22	Grasses		1.5 Berm, Cattle allowed to graze, causing patches	1605391	5418134
Undocumented	MSG3	Wai-iti	MSG	1.5	2.3	12	Blackberry	1.6	26		Fence at toe	Dirt Road and downstream toe	1605833	5418524
Undocumented	MSG4	Wai-iti	MSG	0.4	0.9	21	Grasses	1.4	27	Grasses		Profile: see sketch	1605976	5418890
Undocumented	MSG5	Wai-iti	MSG	1.1	1.5	37	Scrub	1	22	Grass, small broom			1606313	5419127
Undocumented	Con1	Wairoa	Confluence	0.8	0.8	18	New Planting	0.6	22	Grasses, New Planting			1609466	5421351
Undocumented	Con2	Wairoa	Confluence	2.4	3	shear	Grasses	2.9	shear	Grass patches, dill, broom			1609635	5421581
Undocumented	Con3	Wairoa	Confluence	1.8	2.3	14	Scrub	0.5	37	Grasses, Trees		Access Road, drops down to chip road	1609706	5421947
Undocumented	Con4	Wai-iti	Confluence	6	2.1	25	Impenetrable	1.1	19	Grasses, Gorse, Large Shrubs	Fence at toe	Farm Road dirt	1609558	5422459
Undocumented	Con5	Wai-iti	Confluence	2.3	0.7	19	Impenetrable	0.9	24	Grasses, Gorse, Large Shrubs	Cow Excrement and hoof Prints, Fence at Downstream Toe, Land is terraces		1609462	5422206
Undocumented	Con6	Wai-iti	Confluence	1.9	1.8	28	Grasses	2	0	Grasses			1608994	5421582
Council	C	Wai-iti	Council	3	2.5	24	Grassed	2.9	29	Grasses			1608599	5421172

Status	Indictor	River	Stopbank	Crest Width (m)	Upstream Height (m)	Upstream Slope (°)	Upstream Cover	Downstream Height (m)	Downstream Slope (°)	Downstream Cover	Structures	Comments	Easting NZGD 2000	Northing NZGD 2000
Council	C2	Wai-iti	Council	3	2.4	24	Grassed	2.5	22	Grassed	Fence at toe, Powerline perpendicular	2m access road at end, south side is lower, farm road is chip	1609027	5421714
Council	C3	Wai-iti	Council	3.2	2.2	29	Grasses	2.5	28		Fence at toe	Farm Road with chip	1609201	5421953
Council	C4	Waimea	Council	3.1	2.3	23	Grasses	2.4	26				1609632	5423166
Council	C5	Waimea	Council								Fence at toe	Farm Road	1609675	5423318
Council	C6	Waimea	Council	3.1	1.9	26	Grasses	2	25	Grass		Access Road has slight Dip, Terraced at Access Road, Big Power pole	1609771	5423795
Council	C7	Waimea	Council	3	4	27	Grasses	3	22		Fence at toes	Farm Road Chip, Landslip at toe	1611245	5427176
Council	C8	Waimea	Council	3.5	3.7	29	Grasses	3.1	24	Grasses			1611434	5427424
Council	C9	Waimea	Council	3.7	2	30	Grasses	2.9	26	Grasses	Pump station, Powerline at Downstream	Access road 10%	1611771	5428021

Appendix B: CONDITION ASSESSMENT IMAGERY

B.1. Council Features

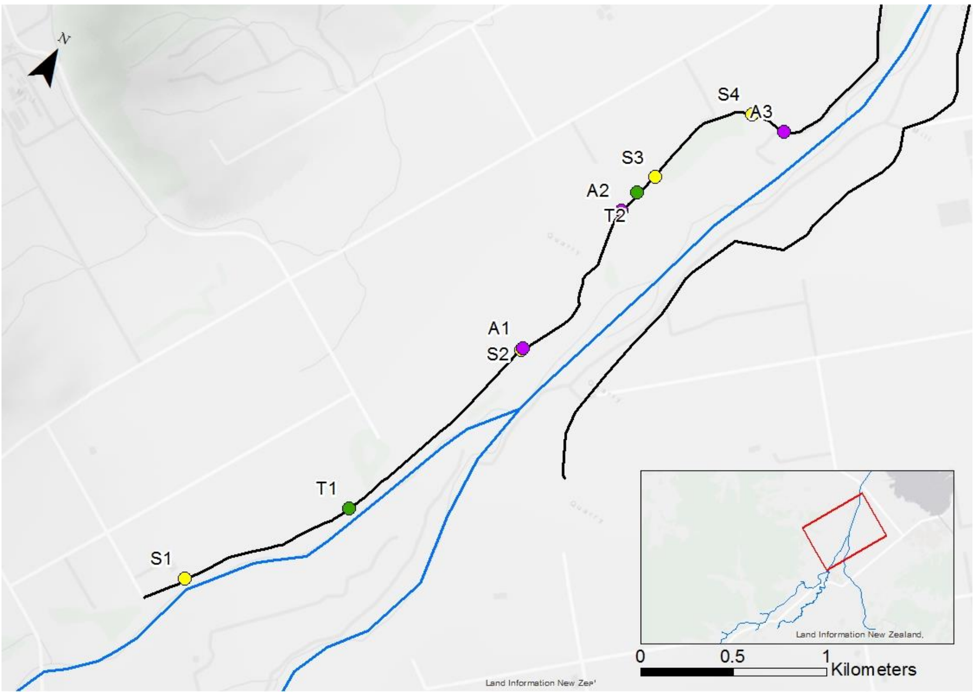


Figure B.1. Upstream Council features map

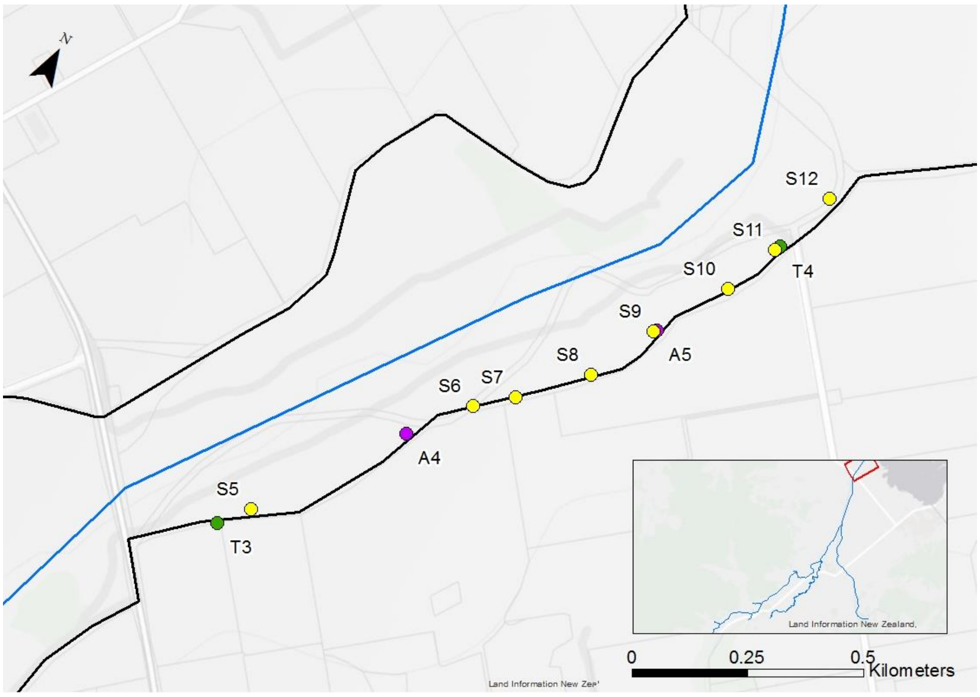


Figure B.2. Downstream Council features map

Table B.1. Council features and details

Indicator	Description	Comment	Easting NZGD 2000	Northing NZGD 2000
A1	Access Road	Across stopbank	1609673	5423320
A2	Access Road	Across stopbank	1609762	5424232
A3	Access Road	Across stopbank	1610312	5425037
A4	Access Road	Across stopbank	1611254	5427221
A5	Access Road	Across stopbank	1611611	5427683
S1	Pump	Small pump landward toe	1608713	5421335
S2	Powerline	In Stopbank	1609668	5423307
S3	Conduit	Flap structure	1609832	5424479
S4	Powerline	In Stopbank	1610112	5425033
S5	Powerline	Waterside of Stopbank	1611045	5426913
S6	Discharge Point	In Stopbank	1611348	5427344
S7	Pump station	In Stopbank	1611418	5427406
S8	Pump station	In Stopbank	1611534	5427529
S9	Pump station	In Stopbank	1611604	5427677
S10	Pump station	In Stopbank	1611697	5427837
S11	Pump station	In Stopbank	1611744	5427961
S12	Pump station	In Stopbank	1611791	5428115
T1	Tree	In Stopbank	1609295	5422105
T2	Tree	In Stopbank	1609786	5424355
T3	Tree	0.15 m roots in crest	1610998	5426852
T4	Tree	Waterside of Stopbank	1611744	5427961

Figure B.3. Access Road at A1

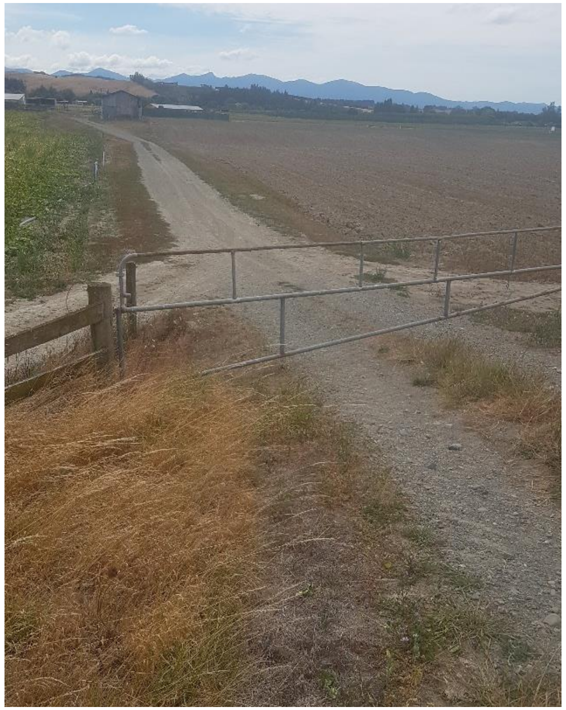


Figure B.4. Access Road at A2

Figure B.5. Access Road at A3

Figure B.6. Access Road at A4

Figure B.7. Access Road at A5

Figure B.8. Pump at S1

Figure B.9. Powerline at S2

Figure B.10. Conduit at S3

Figure B.11. Powerline at S4

Figure B.12. Powerline at S5

Figure B.13. Discharge at S6

Figure B.14. Pump station at S7

Figure B.15. Pump station at S8

Figure B.16. Pump station at S9

Figure B.17. Pump station at S10

Figure B.18. Pump station at S11

Figure B.19. Pump station at S12

Figure B.20. Tree at T1

Figure B.21. Tree at T2

Figure B.22. Tree roots at T3

Figure B.23. Tree at T4

B.2. MSG Features

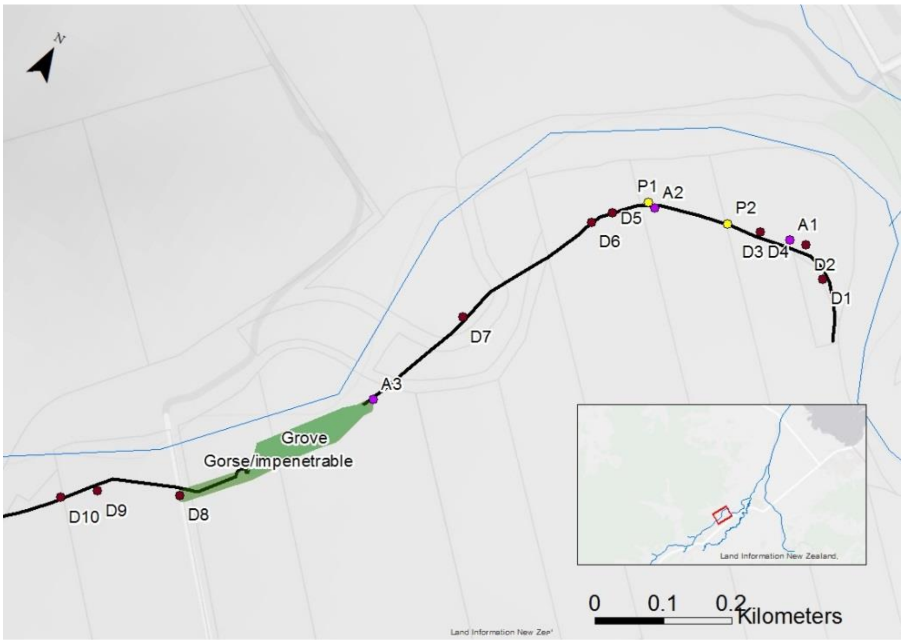


Figure B.24. Downstream MSG features map

Figure B.25. Upstream MSG features map

Table B.2. MSG features and details

Indicator	Description	Comment	Easting NZGD 2000	Northing NZGD 2000
A1	Access Road		1606293	5419142
A2	Access Road		1606028	5419019
A3	Access Road		1605881	5418634
D1	Depression	0.5 m	1606364	5419117
D2	Depression/Slump	0.5 m	1606318	5419148
D3	Depression/Rut	0.6 m	1606250	5419130
D4	Depression/Slump	0.2 m	1606250	5419130
D5	Depression/Rut	0.3 m	1606048	5419047
D6	Depression	0.7 m	1606028	5419019
D7	Depression	0.5 m	1605934	5418804
D8	Exposed Fill	0.3 m	1605706	5418371
D9	Exposed Fill	2 m	1605597	5418316
D10	Depression/Rut	0.4 m	1605555	5418281
D11	Depression	0.5 m	1604570	5417517
D12	Depression	0.7 m	1604570	5417517
P1	Powerline	Along Stopbank	1606085	5419086
P2	Powerline	Along Stopbank	1606202	5419117

Figure B.26. Access Road at A1

Figure B.27. Access Road at A2

Figure B.28. Access Road at A3

Figure B.29. Depression at D1

Figure B.30. Depression/slump at D2

Figure B.31. Depression/rut at D3

Figure B.32. Depression/slump at D4

Figure B.33. Depression/rut at D5

Figure B.34. Depression at D6

Figure B.35. Depression at D7

Figure B.36. Exposed fill at D8

Figure B.37. Exposed fill at D9

Figure B.38. Depression/rut at D10

Figure B.39. Depressiont at D11

Figure B.40. Depression at D12

Figure B.41. Powerline at P1

Figure B.42. Powerline at P2

B.3. Pitfure Features

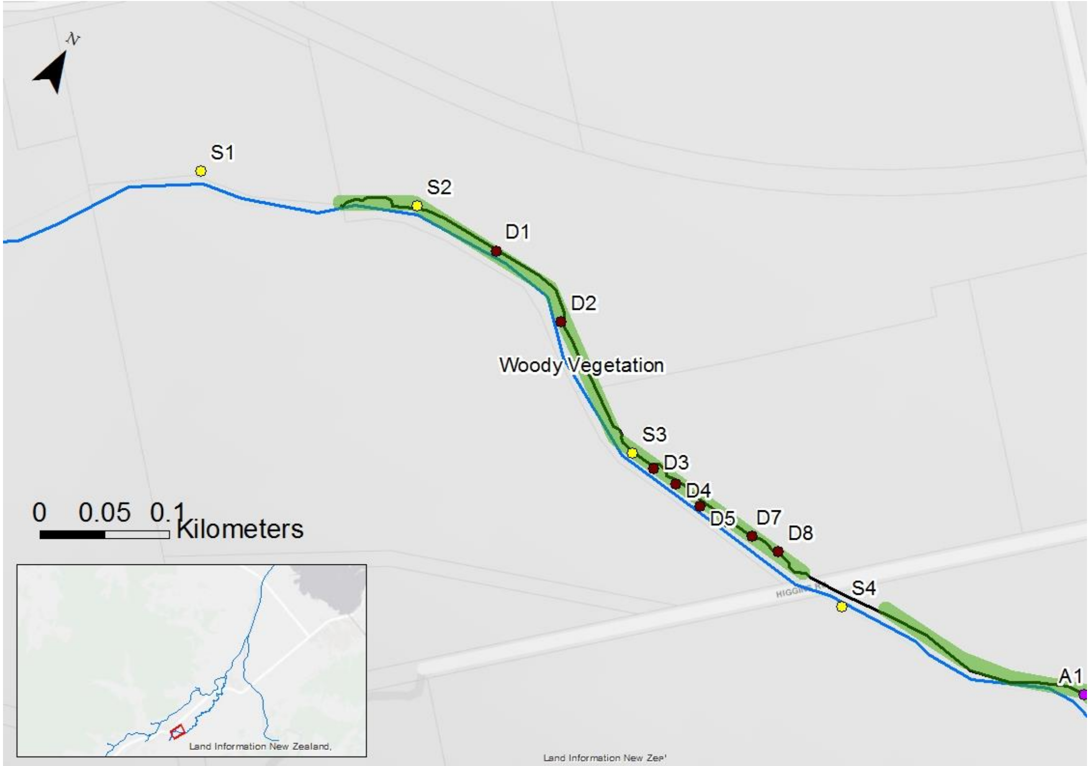


Figure B.43. Downstream Pitfure features map

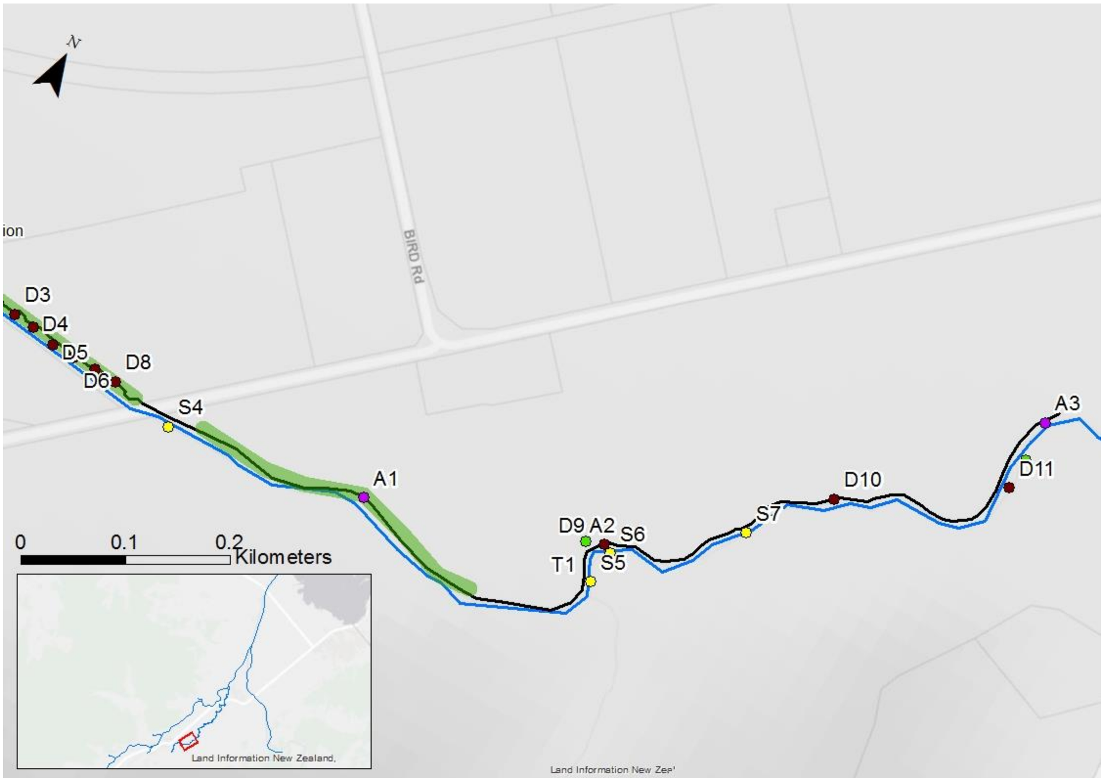


Figure B.44. Upstream Pitfure features map

Table B.3. Pitfure features and details

Indicator	Description	Comment	Easting NZGD 2000	Northing NZGD 2000
A1	Access Road		1605391	5416365
A2	Access Road		1605613	5416441
A3	Access Road		1605920	5416752
D1	Depression/Rut	0.6 m	1604826	5416434
D2	Depression/Rut	0.3 m	1604897	5416412
D3	Void	1.2 m	1605016	5416350
D4	Void	0.6 m	1605037	5416348
D5	Void	0.7 m	1605062	5416342
D6	Void	1.2 m	1605085	5416342
D7	Void	0.6 m	1605108	5416342
D8	Depression	0.5 m	1605131	5416342
D9	Depression/Rut	0.3 m	1605613	5416441
D10	Scour/Depression	0.5 m	1605782	5416589
D11	Slump/Depression	0.6 m	1605921	5416681
S1	Culvert		1604597	5416374
S2	Pipe Bridge		1604756	5416434
S3	Pipe Bridge		1604995	5416352
S4	Blockage	Fallen Tree	1605196	5416330
S5	Pipe Bridge/Scour		1605619	5416404
S6	Blockage/Scour	Sheet Metal Blockage	1605622	5416437
S7	Pipe Bridge/Scour		1605725	5416519
T1	Tree		1605596	5416435
T2	Tree/Scour		1605921	5416712

Figure B.45. Access road at A1

Figure B.46. Access road at A2

Figure B.47. Access road at A3

Figure B.48. Depression/ruts at D1

Figure B.49. Depression/ruts at D2

Figure B.50. Void at D3

Figure B.51. Void at D4

Figure B.52. Void at D5

Figure B.53. Void at D6

Figure B.54. Void at D7

Figure B.55. Depression at D8

Figure B.56. Depression/ruts at D9

Figure B.57. Scour/depression at D10

Figure B.58. Slump/depression at D11

Figure B.59. Culvert at S1

Figure B.60. Pipe bridge at S2

Figure B.61. Pipe bridge at S3

Figure B.62. Fallen tree blockage at S3

Figure B.63. Pipe bridge and scour at S4

Figure B.64. Sheet metal blockage and scour at S5

Figure B.65. Pipe bridge and scour at S6

Figure B.66. Tree at T1

Figure B.67. Tree and scour at T2

B.4. Confluence Features

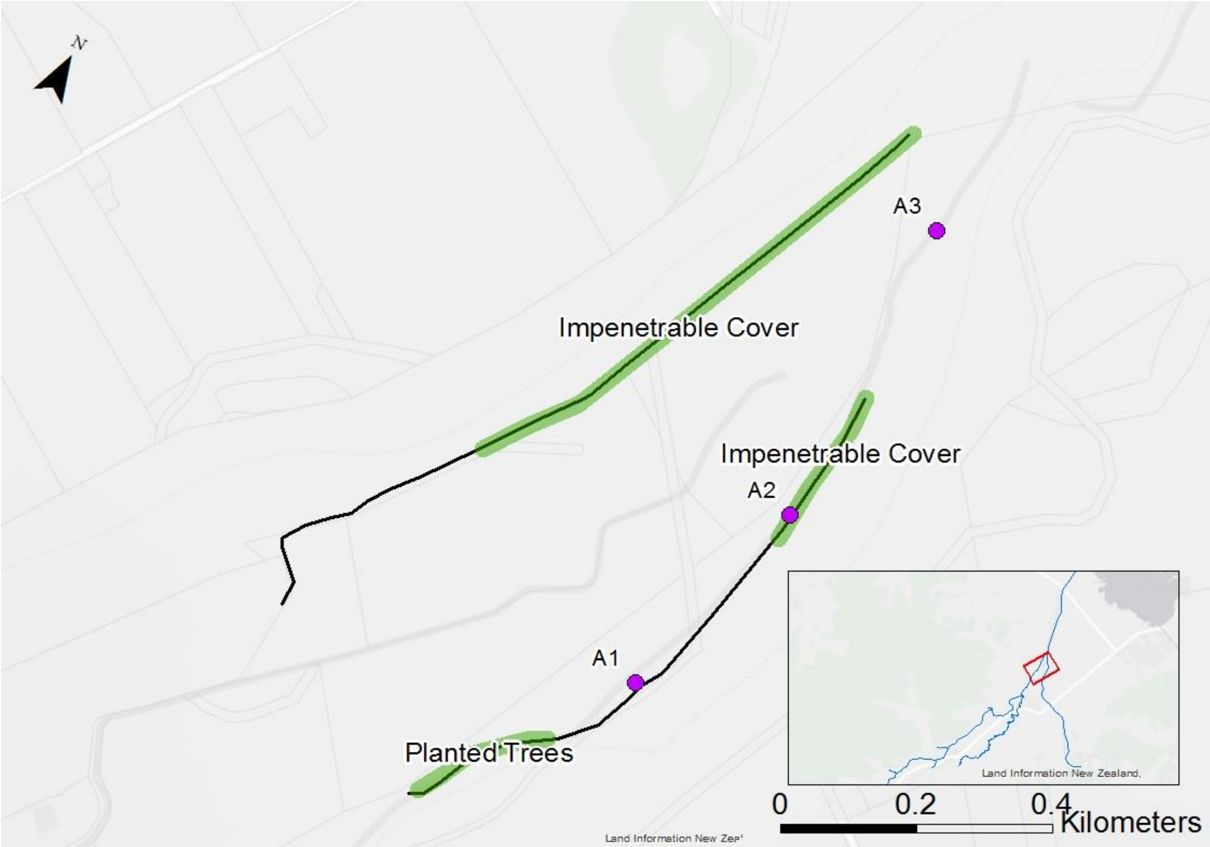


Figure B.68. Confluence features map

Table B.4. Confluence features and details

Indicator	Description	Easting NZGD 2000	Northing NZGD 2000
A1	Access Road	1609630	5421598
A2	Access Road	1609703	5421927
A3	Access Road	1609678	5422400

Figure B.69. Access Road at A1

Figure B.70. Access Road at A2

Figure B.71. Access Road at A3